PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 66 MAY, 1940 No. 5

TECHNICAL PAPERS

AND

DISCUSSIONS

Published monthly, except July and August, at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York, N. Y. Reprints from this publication may be made on condition that the full title of Paper, name of Author, page reference, and date of publication by the Society, are given.

Entered as Second-Class Matter, September 23, 1937, at the Post Office at Lancaster, Pa., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy.

CURRENT PAPERS AND DISCUSSIONS

Paper	Published	Discussion closes
Design of Dowels in Transverse Joints of Concrete Pavements. *Bengt F. Friberg*	Nov., 1938	
Discussion	Mar., June, 1939, May, 1940	Closed
Stress Distribution Around a Tunnel. *Raymond D. Mindlin*	Apr., 1939	
Discussion	Oct., 1939, Feb., 1940	Closed*
Reconstruction of the Walpole-Bellows Falls Arch Bridge. *H. E. Langley and Edward J. Ducey*	Apr., 1939	
Discussion	Sept., Oct., 1939, Jan., 1940	Closed*
Flash-Board Pins. *Chilton A. Wright and Clifford A. Betts*	May, 1939	
Discussion	Nov., Dec., 1939, Jan., Apr., 1940	Closed*
Tension Tests of Large Riveted Joints. *Raymond E. Davis, Glenn B. Woodruff, and Harmer E. Davis*	May, 1939	
Discussion	Sept., Oct., 1939, Apr., May, 1940	Closed*
Combining Geodetic Survey Methods with Cadastral Surveys. *Carl M. Berry*	Sept., 1939	
Discussion	Dec., 1939, Jan., Feb., 1940	Closed*
An Improved Method for Adjusting Level and Traverse Surveys. *Clarence Norris and Julius L. Speert*	Oct., 1939	
Discussion	Jan., Feb., Mar., 1940	Closed*
The Unit Hydrograph Principle Applied to Small Water-Sheds. *E. F. Brater*	Sept., 1939	
Discussion	Jan., Feb., Apr., 1940	Closed*
Development of the Colorado River in the Upper Basin. *Thomas C. Adams*	Sept., 1939	
Discussion	May, 1940	Closed*
Field Tests of a Shale Foundation. *August E. Niederhoff*	Sept., 1939	
Discussion	Jan., Feb., 1940	Closed*
General Wedge Theory of Earth Pressure. *Karl Terzaghi*	Oct., 1939	
Discussion	Jan., Feb., Apr., 1940	Closed*
Functional Design of Flood Control Reservoirs. *C. J. Posey and Fu-Te I*	Oct., 1939	
Discussion	Dec., 1939, Mar., Apr., May, 1940	Closed*
Sewage Disposal Project of Buffalo, New York. *Samuel A. Greeley*	Oct., 1939	
Discussion	Nov., 1939, Apr., May, 1940	Closed*
Relation of the Statistical Theory of Turbulence to Hydraulics. *A. A. Kalinske*	Oct., 1939	
Discussion	Jan., Feb., Mar., Apr., May, 1940	Closed*
Effective Moment of Inertia of a Riveted Plate Girder. *Scott B. Lilly and Samuel T. Carpenter*	Oct., 1939	
Discussion	Dec., 1939, Jan., Feb., Mar., May, 1940	May, 1940
Problems and Trends in Activated Sludge Practice. *Robert T. Regester*	Nov., 1939	
Discussion	Mar., Apr., 1940	May, 1940
Bridge and Tunnel Approaches. *John F. Curtin*	Nov., 1939	
Discussion	Mar., Apr., 1940	May, 1940
Trend in Hydraulic Turbine Practice: A Symposium	Nov., 1939	
Discussion	Jan., Mar., Apr., May, 1940	May, 1940
Effects of Rifling on Four-Inch Pipe Transporting Solids. *G. W. Howard*	Nov., 1939	
Discussion	Mar., Apr., May, 1940	May, 1940
Transient Flood Peaks. *Henry B. Lynch*	Nov., 1939	
Discussion	Jan., Feb., Mar., Apr., May, 1940	May, 1940
The Rôle of the Engineer in Air Sanitation: A Symposium	Nov., 1939	
Discussion	Feb., 1940	May, 1940
Channelization of Motor Traffic. *Guy Kelcey*	Dec., 1939	
Discussion	Mar., Apr., May, 1940	May, 1940
Water Supply on Upper Salt River, Arizona. *John Girand*	Dec., 1939	
Discussion	Mar., Apr., 1940	May, 1940
Analysis of Legal Concepts of Subflow and Percolating Waters. *C. F. Tolman and Amy C. Stipp*	Dec., 1939	
Discussion	Feb., Apr., May, 1940	May, 1940
Miniature System of First-Order Alinement and Triangulation Control. *Floyd W Hough*	Dec., 1939	
Discussion	Apr., 1940	June, 1940
Pressure-Momentum Theory Applied to the Broad-Crested Weir. *H. A. Doeringsfeld and C. L. Barker*	Dec., 1939	
Discussion	Mar., Apr., 1940	June, 1940
Norris Dam Construction Cableways. *R. T. Colburn and L. A. Schmidt, Jr.*	Dec., 1939	
Discussion	Mar., Apr., 1940	June, 1940
Standards of Professional Relations and Conduct. *Daniel W. Mead*	Jan., 1940	
Discussion	Mar., Apr., 1940	June, 1940
Theory of Elastic Stability Applied to Structural Design. *Leon S. Moisseiff and Frederick Lienhard*	Jan., 1940	June, 1940
Chicago River Control Works. *H. P. Ramey*	Jan., 1940	June, 1940
Axioms in Roadway Soil Mechanics. *Henry C. Porter*	Feb., 1940	June, 1940
Measuring the Potential Traffic of a Proposed Vehicular Crossing. *N. Cherniack*	Feb., 1940	June, 1940
Sealing the Lagoon Lining at Treasure Island with Salt. *Charles H. Lee*	Feb., 1940	June, 1940
Progress Report of Committee of the Hydraulics Division on Flood Control	Feb., 1940	
Discussion	Apr., May, 1940	June, 1940
Foundation Experiences, Tennessee Valley Authority: A Symposium	Mar., 1940	
Discussion	May, 1940	July, 1940
Design of Hinges and Articulations in Reinforced Concrete Structures. *George C. Ernst*	Apr., 1940	Aug., 1940
Permissible Composition and Concentration of Irrigation Water. *W. P. Kelley*	Apr., 1940	Aug., 1940
Progress Report of the Committee on Flood-Protection Data	Apr., 1940	Aug., 1940
Progress Report of the Committee of the Structural Division on Oxygen Cutting (Flame Cutting) of Structural Steel	Apr., 1940	Aug., 1940

NOTE.—The closing dates herein published are final except when names of prospective discussers are registered for special extension of time.

* Publication of closing discussion pending.

CONTENTS FOR MAY, 1940

PAPERS

DISCUSSIONS

CONTENTS FOR MAY, 1940 (*Continued*)

For Index to all Papers, the discussion of which is current in PROCEEDINGS, *see page 2*

The Society is not responsible for any statement made or opinion expressed in its publications

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

MASONRY DAMS

A SYMPOSIUM

NOTE.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by September 15, 1940.

FOREWORD

BY THE AUTHORS

Engineering progress made in connection with the design and construction of dams of unprecedented height—such as Boulder Dam on the Colorado River near Las Vegas, Nev., Grand Coulee Dam on the Columbia River in central Washington, and Shasta Dam on the Sacramento River in north-central California—has thrown much light on what have heretofore been somewhat uncertain problems in the science of dam building. Extensive researches, conducted in the field as well as in the office and laboratory, have developed important information regarding the exploration of foundation conditions, ways of predicting foundation deformations, methods of analyzing structural action, means of determining internal stress conditions, fundamental laws regarding heat flow, and its control in mass-concrete structures, and other basic criteria involved in the more technical phases of dam design. Similar investigations, conducted in both laboratory and field, have developed pertinent data regarding the best methods of preparing foundation formations, selecting and proportioning concrete ingredients, placing and curing mass concrete, controlling internal temperature changes, and conducting other operations of a construction nature. In view of these developments it seemed desirable to offer a carefully planned symposium on masonry dams for general discussion.

BASIC DESIGN ASSUMPTIONS

BY IVAN E. HOUK,[1] AND KENNETH B. KEENER,[2] MEMBERS, AM. SOC. C. E.

SYNOPSIS

Basic assumptions and related technical considerations involved in the design of high and important masonry dams of the single-arch, curved gravity, and straight gravity types, built on rock foundations, are presented in this paper. Some of the statements are applicable to the design of other types of masonry dams. However, this paper does not attempt to cover, comprehensively, the fundamental criteria involved in the design of multiple-arch dams, reinforced-concrete slab and buttress dams, roundhead buttress dams, or other special types of masonry dams treated in more detail in other papers of the Symposium.

Furthermore, this paper is confined, primarily, to the basic assumptions which have either undergone appreciable modifications during recent years or have been developed as entirely new criteria for the design of important masonry dams of the aforementioned types. Basic information, such as normal stream-flow conditions at the site, magnitudes of ice pressure, maximum range of seasonal concrete temperature changes, physical properties of concrete materials, and other data which may be intelligently ascertained or predicted from readily available records, or may be determined by routine laboratory measurements, are not discussed. Assumptions involved in determining maximum anticipated flood intensities for use in designing spillway features constitute a special problem and consequently are not included.

Basic assumptions for the design of masonry dams are treated from the viewpoints of the dam site, the dam, load conditions, structural action, stability factors, and stress conditions. Details of procedures involved in dam design are not included. However, a bibliography at the end of the paper lists the more important recent articles which should be consulted by any one interested in an exhaustive consideration of the subject.

THE DAM SITE

The adequacy of the site constitutes the first fundamental criterion for the design of a masonry dam. A gravity structure is only as stable as the base on which it rests. An arched structure is only as stable as the walls against which

[1] Senior Engr., Technical Investigations, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

[2] Designing Engr. on Dams, Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

the arch elements abut. The stability of the rock formations, which constitute the site and which, therefore, must support the dam and the loads transmitted by the dam, depends on the physical properties of the rock, the uniformity of the rock structure, the arrangement of the geological formations, the presence of sheared or faulted zones, the presence of seams or other bodies of disintegrated, unsatisfactory material, and the extent to which inadequate characteristics of the geological structure may be corrected by feasible foundation improvements.

Surface indications of bedrock conditions are often misleading. At many dam sites natural forces have materially altered the foundation formations to appreciable depths below the exposed surfaces. Consequently, the adequacy of the site must be determined by thorough explorations into the geological strata to such depths as may be affected by the construction of the dam and the subsequent filling of the reservoir. Cores of comparatively small diameter, obtained by drilling operations, may be sufficient in the case of unusually uniform and satisfactory rock formations. However, such information has sometimes been misleading or has had a tendency either to discredit the foundation or to confuse the interpretation of the data. As a general rule, the larger the diameter of the cores extracted from the rock, the more reliable is the information obtained and the greater the significance that can be attributed to the specimens. The most satisfactory methods of foundation explorations include the excavation of tunnels into the canyon walls, the sinking of shafts into the foundation strata, and the removal of large-diameter cores (all three of which permit visual inspection of the rock in place), in addition to the drilling and removal of numerous cores of relatively small diameters which do not permit visual inspection of the geological formations (1).[3]

From the viewpoint of conditions at the dam site the basic assumptions involved in the design of important masonry dams may be briefly listed as follows:

1. The rock which constitutes the foundation and abutments at the site is strong enough to carry the forces imposed by the dam with stresses well below the elastic limit at all places along the contact planes;
2. The bearing power of the geologic structure along the foundation and abutments is great enough to carry the total loads imposed by the dam without rock movements of detrimental magnitude;
3. The rock formations are homogeneous and uniformly elastic in all directions, so that their deformations may be predicted satisfactorily by calculations based on the theory of elasticity, by laboratory measurements on models constructed of elastic materials, or by combinations of both methods; and
4. The flow of the foundation rock under the sustained loads which result from the construction of the dam and the filling of the reservoir may be adequately allowed for by using a somewhat lower modulus of elasticity than would otherwise be adopted for use in the technical analyses.

Although it is of rather an elemental nature, assumption 1 is receiving more and more attention as dams are continually being planned for higher heads and

[3] Numerals in parentheses, thus: (1), refer to corresponding items in the Appendix.

higher working stresses. In the early days of dam construction, when dam cross sections were planned for maximum working stresses of about 10 or 12 tons per sq ft, the adequacy of the foundation rock from the stress viewpoint was often determined arbitrarily by geological inspection and engineering judgment. Today, when dam cross sections are being planned for maximum working stresses of 40 or 50 tons per sq ft, samples of the actual rock are taken from the foundations and abutments and tested in the laboratory, where the strength can be measured accurately. Consequently, this criterion, which has been listed as an assumption, and which formerly was often in reality merely an assumption, is now only an assumption in so far as the question arises as to whether or not the test samples accurately determine the true strength of the rock formations on which the dam is to be constructed. Present engineering practice in designing and constructing important masonry dams requires that an adequate number of representative samples of the foundation and abutment rock be taken and properly tested in the laboratory, so that the matter of adequate rock strength may be classified as measurable fundamental data rather than as a basic assumption.

Assumption 2 is true if assumption 1 is true and if the rock formations are continuous, or can be made satisfactorily continuous by grouting operations or by other methods of foundation preparation, such as washing out seams of soft material and refilling with grout, excavating unsatisfactory rock and replacing with concrete, and other measures of a precautionary nature.

Assumption 3 is somewhat open to question. It is doubtful if the rock formations at any dam site are ever sufficiently free from cracks, seams, fissures, joints, and bedding planes to be considered uniformly homogeneous. It is also doubtful if such formations are ever uniformly elastic in all directions, so that their movements under loads transmitted by the dam can be accurately calculated by formulas based on the theory of elasticity—formulas which, at best, can be considered only approximate. Furthermore, it is doubtful if methods of foundation preparation, such as the aforementioned, can ever produce an ideally homogeneous and uniformly elastic rock structure, even if they are entirely adequate to permit the construction of a safe masonry dam. However, for the present, elastic formulas, together with measurements on elastic models, constitute the only available methods of estimating the magnitude of foundation and abutment deformations. These movements, although small, may be relatively large proportions of the total dam deflections. Consequently, they constitute quantities that should be considered in the technical analyses.

Assumption 4 is probably within reasonable limits of accuracy in all cases where assumptions 1 and 2 are fulfilled. In connection with the consideration of foundation and abutment movements, it is interesting to note that measurements at the 313-ft Ariel Dam, a constant-angle arch dam on the north fork of Lewis River, southern Washington, showed rock movements at the south abutment about twice as great as those calculated in the trial load analyses (2). This difference between actual and anticipated abutment movements at Ariel Dam may have been partly due to flow, inaccuracies in determining the modulus of elasticity, or approximations in the elastic formulas. However, it is possible that the difference may have been due entirely to slippage of the

geological formations along seams, cracks, fissures, or other planes of weakness not sufficiently strengthened in the abutment preparations.

The Dam

The masonry dam, as constructed by the most approved methods of today (1940), is a safer, more reliable, and better understood structure than it was twenty, or even ten, years ago. Modern developments in manufacturing special cements, practical refinements in controlling the quality, proportioning, and mixing of concrete ingredients, and continued progress in improving mechanical equipment and construction procedures have eliminated many heretofore sources of uncertainty. These advances have gone hand in hand with the evolution of design methods. They are reflected in both purchase and construction specifications. Their effects are evidenced by betterments in the quality and economy of construction which characterize the more recent structures. Perhaps the most noteworthy aspects of these accomplishments are the closer approach to uniformity of the concrete in place, so that now the designer may consider concrete, more than ever before, as a satisfactory and dependable material for use in constructing important masonry dams.

The inherent limitations of concrete in respect to volume constancy have been largely offset by the introduction and use of special cements of low-heat, modified, or puzzolan-portland types (3), by building massive structures in smaller blocks or units, by artificially cooling the concrete to stable thermal conditions when necessary, and by grouting the separate units together to form a finished monolithic structure (4). The location and spacing of the contraction joints that form the boundaries of the units are determined by positions of foundation irregularities; locations of penstocks, outlets, and sluiceways; and the maximum size of individual blocks within which cracks from volume changes may not be expected to occur. The joints are filled with cement grout, injected under pressure. Cores taken from completed dams, as well as extensive laboratory investigations, have shown that the grout actually penetrates to all parts of the joint areas. In some cases the penetration was so satisfactory that joint locations could scarcely be identified in cores taken across the joints. Escape of liquid grout and leakage of water from the joints are prevented by flexible strips of non-corrodible sheet metal embedded in the concrete on both sides of the joints.

The use of economical lean mixes of low slump, containing large-size aggregate, has made uniformity of all ingredients and close control of concrete manufacture imperative. More care is continually being exercised in designing mixtures, selecting and processing aggregates, conducting mixing operations, conveying the product, and placing the concrete in the dam. More rigid control of sand gradation is being obtained by fractionation in classifiers and recombination, as at Grand Coulee Dam, in Washington, or by the addition of fine blending sand. At Marshall Ford Dam, in Texas, the equivalent of a blending sand was produced by reduction of excess coarse sand in rod mills. Segregation and breakage of coarse aggregate are avoided by more careful methods of handling and storing. Better stabilization of moisture content is obtained by providing ample drainage facilities at the aggregate stock piles,

particularly in the case of sand storage. More satisfactory proportioning and mixing of ingredients are obtained by using improved weighing batchers, water meters, automatic batch recorders, sequence charging of mixers, and mixer blade designs of more appropriate shape and arrangement. The poor adaptability of the "torque-type" consistency meter for registering changes in consistency of stiff mass concrete has been overcome at Grand Coulee Dam by the development of a meter that depends for its operation on the relation between the consistency of the batch and its tendency to tilt the mixer on the trunnions.

Improvements in conveying concrete have included the substitution of buckets, pumps, and other preferred methods of transportation for belt conveyers and chuting systems. More satisfactory compaction in placing is being secured by internal vibration whenever possible. Construction joint surfaces are being prepared more carefully before adding new concrete, so as to insure higher bonding strength along contact planes.

Considering the most approved methods of constructing masonry dams, as well as the aforementioned developments in methods of manufacturing, transporting, and placing concrete, the principal basic assumptions involved in the design of important masonry dams, from the viewpoint of the physical properties of the dam, may be listed briefly as follows:

5. The base of the dam is thoroughly keyed into the rock formations along the foundation and abutments;

6. Construction operations are conducted so as to secure satisfactory bond between the concrete and rock materials at all areas of contact along the foundation and abutments;

7. The concrete in the dam is homogeneous in all parts of the structure;

8. The concrete is uniformly elastic in all parts of the structure, so that deformations due to applied loads may be calculated by formulas derived on the basis of the theory of elasticity, or may be estimated from laboratory measurements on models constructed of elastic materials;

9. Effects of flow of concrete may be adequately allowed for by using a somewhat lower modulus of elasticity under sustained loads than would otherwise be adopted for use in the technical analyses;

10. Construction joints are properly grouted under adequate pressures, or open slots are properly filled with concrete, so that the dam may be considered to act as a monolith; and

11. Sufficient drains are installed in the dam to reduce such uplift pressures as may develop along areas of contact between the concrete and rock materials.

Assumptions 5 to 11, inclusive, need little comment. Assumption 5, regarding the keying of the dam into the rock formations, should always be feasible. Assumption 6, regarding the securing of satisfactory bond along areas of contact between the masonry and the rock, may be difficult of attainment at sites where the geological strata consist of shale or other not wholly satisfactory formations. In such cases sufficient stability may be assured by keying the dam into the rock to more than usual depths. Assumptions 7, 8, and 9 are believed to be within reasonable limits of accuracy in the case of the most approved methods of construction used in the United States. They

may be somewhat uncertain in the case of some foreign masonry dams where different mixtures of concrete are used in different parts of the dam. These assumptions were usually more or less open to question during the early history of masonry dam construction. However, modern researches in methods of proportioning ingredients, mixing, conveying, and placing mass concrete have eliminated many of the uncertainties regarding the use of concrete as a structural material. Assumptions 10 and 11, regarding the grouting of construction joints, filling of open slots, and installation of foundation drains, are matters of design and construction which should always be obtainable.

Load Conditions

Load conditions at masonry dams may be classified as "normal" and "abnormal." Normal dead loads include concrete weights, weights of gates, bridges, or other superimposed features, and weights of such materials as may be deposited on the sloping faces of the dam. Normal live loads include reservoir-water pressures, tailwater pressures, uplift forces, and forces caused by seasonal changes in concrete temperature. Abnormal loads, the most important of which are all of a live-load nature, may include ice pressures, increases in fluid pressures during silt flows, earthquake shocks, construction loads, and any other applied loads of unusual or temporary nature. Construction loads include forces produced during grouting operations, internal stresses caused by artificial cooling operations, and internal stresses caused by the generation of chemical heat during the curing of the concrete. In locations where severe climatic conditions occur during the winter months, and where relatively high reservoir surfaces are maintained at such times, ice pressures may be considered as normal live loads rather than as abnormal loads.

Water pressures may be calculated accurately for given water-surface levels. Unit concrete weights may be determined accurately from laboratory samples. Weights of features superimposed on the dam, weights of materials deposited on the slopes of the dam, seasonal concrete temperature changes, and maximum ice pressures may be satisfactorily predicted from available records. Consequently these load conditions, the first two of which usually constitute the most important normal loads at masonry dams of massive arch, arched gravity, and gravity types, need not be discussed herein. In the case of high storage dams such ice loads as may develop during the winter months usually occur at elevations considerably lower than the top of the dam so that the ice pressures, together with the water pressures caused by the accompanying partial-depth reservoir loads, are usually less severe loading conditions than those existing when the reservoir is filled to the top of the dam.

Some questions have arisen in the past regarding increases in fluid pressures which may occur due to the presence of large proportions of silt, sand, or gravel in flood flows reaching the dam. The Bullards Bar Dam, a power and debris-control dam built on the North Fork of the Yuba River in California, in 1924, was designed for a fluid pressure of 90 lb per cu ft (5). Measurements at the Miami Conservancy District flood-control dams of southwestern Ohio (6), and also at the Tieton irrigation supply dam of central Washington (7), showed that horizontal pressures in the sand and gravel portions of the embankments did not

exceed hydrostatic pressures, that silt materials in the cores consolidated within comparatively short times after deposition, and that horizontal pressures in the silt deposits decreased to hydrostatic values during the periods of consolidation.

In the case of a relatively high storage dam, the greater portion of the debris load is deposited at the upper end of the reservoir so that only comparatively small quantities of silt reach the dam where they may produce increases in horizontal pressure. These increases in horizontal pressure occur only during the passage of the flood and during the period of consolidation, which begins as soon as the flood subsides. Consequently, unless floods occur in close succession, the depths at which increased fluid pressures may be exerted against the upstream face of the dam are relatively small proportions of the total height of the structure. Analyses for several high masonry storage dams have shown that effects of increased horizontal pressures caused by silt conditions are negligible in comparison with effects of earthquake shocks. In designing relatively low diversion dams on streams carrying large proportions of silt, it may sometimes be desirable to consider fluid pressures greater than those caused by clean water.

Recent investigations of load conditions at masonry dams have considered uplift forces along areas of contact between the dam and the rock formations, uplift pressures at construction joints and within the pores of the concrete, forces due to abnormal changes in concrete temperature, forces produced during grouting operations, and abnormal loads caused by the occurrence of severe earthquake shocks.

Measurements at existing masonry dams have shown that uplift forces develop at the base of the dam, even if the foundation grouting operations have been unusually thorough and extensive and although numerous foundation drains have been provided beneath the upstream portion of the dam. However, measurements at Norris, Owyhee, and Boulder dams have shown that such uplift forces may be materially reduced by the installation of additional drains at the areas where the uplift pressures develop. Consequently, in planning masonry dams for sites where satisfactory rock foundations exist, and where approved grouting and draining provisions are specified, it is not necessary to design the cross sections for full uplift pressures acting over the entire area of the base. Uplift investigations conducted during the six years 1933–1939 have shown no necessity for changing the conclusion previously expressed—namely, that it seldom is necessary to apply a straight-line pressure distribution from reservoir head to tailwater head to more than two thirds the area of the base (8) (9). Of course, unusually severe uplift conditions must be assumed in designing masonry dams to be built on porous sandstone foundations, shale beds, or other not wholly satisfactory geological formations. Thus far, measurements at existing masonry dams have shown no serious uplift pressures at construction joints or within the pores of the concrete.

Internal stresses caused by artificial cooling operations may be kept within safe limits by building the dam in blocks or columns, separated by contraction joints as previously mentioned, and grouting the joints after the cooling has been completed (10). Stresses caused by increases in temperature after grouting are compression. Such stresses are beneficial since they tend to tighten the

joints and produce a more monolithic structure. In the case of arch dams they also tend to produce upstream movements that work against the reservoir load. Forces produced by grout pressures during joint grouting operations must be carefully considered in planning the grouting procedure. If the grout is injected under too high pressures, the concrete columns may be cracked by shearing or bending stresses. Furthermore, in the case of arch dams the cantilever sections may be deflected upstream until horizontal cracking occurs along the downstream face. Formulas for calculating the flow of heat in masonry dams were developed during the design of Boulder Dam (11). Studies of seasonal and curing temperature changes in actual dams were published in 1930 and 1931 (12) (13).

The major development in American masonry dam design during the ten years 1929–1939, from the viewpoint of load conditions, has been the consideration of effects produced by severe earthquake shocks. Earthquake effects have been included in designing practically all large masonry dams in Japan since the Kwanto earthquake of September 1, 1923. However, the first important masonry dam in America in which earthquake loads were carefully considered in designing the cross section was the 328-ft Morris Dam, completed by the City of Pasadena, Calif., in 1934 (14). Formulas for analyzing effects of earthquake accelerations on load conditions at masonry dams were developed in connection with the work of the Bureau of Reclamation on the design of the 220-ft Madden Dam on the Chagres River, Panama Canal Zone, completed in 1934, and the 726-ft Boulder Dam, on the Colorado River near Las Vegas, Nev., completed in 1936 (15).

Accelerations caused by earthquakes may affect the stability and internal stress conditions of masonry dams, even if the epicenters are some distances from the dams. When the analyses of earthquake loads were first begun, only horizontal accelerations were considered. More recently effects of vertical as well as horizontal accelerations have been included in the stress and stability analyses. Effects of vertical accelerations are especially important in designing gravity dams where no arch action is available to assist in carrying the increased loads. In connection with the consideration of vertical earthquake vibrations at gravity dams, it should be noted that the maximum downward acceleration, as well as the maximum upward acceleration, must be analyzed. The maximum effect on sliding factors during the full condition of the reservoir is produced when the acceleration is acting downward, whereas the maximum effect on stresses is produced when the acceleration is acting upward. In the case of horizontal accelerations, maximum effects on both sliding factors and stresses during the full condition of the reservoir are produced when the acceleration is acting in an upstream direction.

Earthquake accelerations cause changes in sliding factors because uplift pressures are not assumed to be changed by the earthquake forces.

Considerations of earthquake effects have usually been based on assumptions of maximum accelerations equal to one tenth of gravity, periods of vibration of 1 sec, and horizontal vibrations acting normal to the longitudinal axis of the dam. These assumptions are usually considered to be severe enough without including allowances for such resonance as may occur. Furthermore, some

increases in stress conditions and some encroachments on the factors of safety of masonry dams are usually considered permissible for such temporary and abnormal load conditions, so that actual increases in cross sections required by the consideration of earthquake accelerations are seldom of appreciable magnitude. Available records of earthquake occurrence are not sufficient to justify the conclusion that any section of the United States may not experience such phenomena sometime in the future.

From the viewpoint of load conditions, some of the basic assumptions involved in the design of important masonry dams may be listed briefly as follows:

12. Effects of increases in horizontal pressures caused by silt contents of flood waters usually may be ignored in designing high storage dams, but may require consideration in designing relatively low diversion structures;

13. Uplift forces adequate for analyzing conditions at the base of the dam are adequate for analyzing conditions at horizontal concrete cross sections above the base;

14. Internal stresses caused by natural shrinkage and by artificial cooling operations may be adequately controlled by proper spacing of contraction joints;

15. Internal stresses caused by increases in concrete temperature after grouting are beneficial;

16. Maximum pressures used in contraction-joint grouting operations should be limited to such values as may be shown to be safe by appropriate stress analyses;

17. No section of the United States may be assumed to be entirely free from the occurrence of earthquake shocks;

18. Assumptions of maximum earthquake accelerations equal to one tenth of gravity are adequate for the design of important masonry dams without including additional allowances for resonance effects;

19. Vertical as well as horizontal accelerations should be considered, especially in designing gravity dams; and

20. During the occurrence of temporary abnormal loads, such as those produced by earthquake shocks, some increases in stress magnitudes and some encroachments on usual factors of safety are permissible.

Structural Action

The structural action of a masonry dam in transmitting dead and live loads to the foundation and abutments is much better understood and much more carefully analyzed today (1940) than it was a few years ago. Recent researches, model tests, and experimental measurements at existing dams have developed much pertinent information regarding different methods of load transference and proper procedures to follow in analyzing such load transference (16) (17) (18). In this connection special acknowledgments should be made to the excellent investigational work of the Engineering Foundation Arch Dam Committee (19) (20) (21).

Engineers familiar with the latest developments in masonry dam design may now determine, in a fairly accurate manner, the distribution of load between

arch and cantilever elements in thin arch dams, thick arch dams, and massive curved gravity dams. They may also analyze effects of twist and beam action in both straight and curved gravity dams, as well as effects of twist and tangential shear in both arch and arched gravity dams. It is no longer necessary to design an arch dam on the assumption that all live loads are carried by arch action, or to design a curved gravity dam on the assumption that all live loads are carried by gravity action. Furthermore, it is no longer necessary to neglect effects of foundation and abutment deformations in analyzing the structural action of masonry dams. Fairly accurate estimates of the effects of such deformations may now be included in the mathematical calculations for both arch and gravity structures.

The distribution of load in a masonry dam is determined by trial load methods. The fundamental assumption involved in the trial load method is that the load is distributed between the different systems of structural action in such a way that the resulting deflections of the different systems are equal at all conjugate points in the structure. Since the distribution which fulfils this criterion can be determined feasibly only by assuming different distributions and calculating resulting deflections until a satisfactory agreement is reached, the procedure is logically termed the "trial load" method. In an arch dam the different systems of load transfer are the arches and cantilevers. Effects of tangential shear and twist are usually analyzed by applying internal, self-balancing loads and couples to the two systems and bringing deformations into agreement in tangential and angular directions as well as in radial directions. In a gravity dam the different systems of load transfer are the cantilever elements, horizontal beam elements, and the twisted structure.

The development of the trial load method of analyzing masonry dams was begun in the Denver, Colo., office of the Bureau of Reclamation in 1923, about the time a similar procedure was being investigated in Europe. The Bureau's first use of the method in analyzing arch dams (a method which did not include considerations of rock movements, tangential shear, or twist effects) was described in a paper published by the Society in 1929 (22). A complete description of the amplified method now used by the Bureau was published in bulletin form in 1938, as one of the final reports on the Boulder Canyon Project (23).

Trial load analyses of about twenty arch dams have shown that structural effects, not previously considered in masonry dam design, exert appreciable influences on load distribution and stress magnitude. Considerations of rock movements naturally tend to increase the calculated deflections of the structure. Such considerations usually result in somewhat lower arch and cantilever stresses along foundation and abutment locations without material stress changes in the central and upper portions of the structure. The general result of including tangential shear effects in trial load analyses is a decrease in radial deflections near the crown section, an increase in radial deflections near the abutments, and a slight increase in arch stresses at the abutments without appreciable stress changes at the crown. Such effects are not important at narrow canyon sites, as at Shoshone Dam, in Wyoming, but are important at relatively wide sites, as at Gibson Dam, in Montana.

The general effect of considering twist action is a decrease in radial deflec-

tions at practically all locations, a decrease in maximum arch stress, a decrease in cantilever stress at the downstream face of the dam, and an increase in cantilever stress at the upstream face of the dam (24).

Twist and beam effects in gravity dams occur primarily along the sloping abutments, the effects usually increasing in importance as the steepness of the canyon wall increases. Such effects usually cause decreases in inclined cantilever stresses at the downstream face of the dam, increases in inclined cantilever stresses at the upstream face of the dam, horizontal compression stresses along the downstream face at the abutments, and horizontal tension stresses at the upstream face at the abutments. If not relieved by suitable construction procedure, such as the provision of vertical slots as at Grand Coulee Dam, twisting action and beam action may cause diagonal cracking at the downstream face in the higher central portion of the dam near the abutments. A comprehensive description of the use of the trial load method in analyzing effects of twist and beam action in gravity dams, including formulas, was published in England in 1937 (25).

From the viewpoint of structural action, the basic assumptions involved in the design of important masonry dams may be listed briefly as follows:

21. Effects of foundation and abutment deformations should be included in the technical analyses;

22. In monolithic straight gravity dams some proportions of the loads may be carried by twist action and beam action at locations along the sloping abutments, as well as by the more usually considered gravity action;

23. Detrimental effects of twist and beam action in straight gravity dams, such as cracking caused by the development of tension stresses, may be prevented by suitable construction procedure;

24. In monolithic curved gravity and arch dams some proportions of the loads may be carried by tangential shear and twist effects, as well as by the more usually considered arch and cantilever actions; and

25. The distribution of loads in masonry dams may be determined by bringing the calculated deflections of the different systems of load transference into agreement at all conjugate points in the structure.

Stability Factors

Until a few years ago the stability of a straight gravity dam against failure by sliding was usually determined by calculating the ratio of the total horizontal force to the total vertical force, a ratio known as the "sliding factor," and comparing this ratio with the friction coefficient for concrete sliding on concrete or concrete sliding on rock. If the calculated value of the sliding factor was less than the appropriate value of the friction coefficient at all elevations analyzed, it was assumed that the section would be stable and that the additional security provided by keying the concrete base into the rock foundation, together with the bond secured at the contact areas, would furnish adequate factors of safety at all locations. It was recognized that the tendency of the structure to move downstream under the applied reservoir load would be resisted by the shearing strengths of the concrete and rock materials in all cases where dams

were built by approved methods on satisfactory foundation strata; but attempts to evaluate actual factors of safety, including restraining effects of shearing strength, usually were not made. In 1933 the late D. C. Henny, M. Am. Soc. C. E., consulting engineer on many of the dams built by the Bureau of Reclamation, proposed a method for calculating the factor of safety against downstream movement, including allowances for shearing strength (26). Since that time engineers of the Bureau of Reclamation have been considering the stability of straight gravity dams on the basis of the shear-friction factor of safety, a factor which is calculated in essentially the same manner as Mr. Henny's shear safety factor, *Q*. F. D. Scheidenhelm, M. Am. Soc. C. E., made detailed studies of shear resistance in preparing a report on the stability of the State Line Dam on Cheat River, West Virginia, in 1921, but did not publish the results of his investigations.

The basic assumptions involved in calculating shear-friction factors of safety and in using such factors to determine the stability of gravity dams against movement along horizontal planes have been set forth herein. Recapitulating briefly, the principal pertinent assumptions are numbers 3, 5, 6, and 7, regarding the homogeneity of the rock, the keying of the concrete base into the foundation, the securing of satisfactory bond, and the uniformity of the concrete.

The principal uncertainty involved in evaluating the shear-friction factor of safety is the determination of the average shearing strength of the material. The engineer must remember that the proper value to use in the formula is the shearing strength of the rock if the rock is weaker than the concrete, or the shearing strength of the concrete if the concrete is weaker than the rock. Values of shearing strength used in calculating shear-friction factors of safety for dams built by the Bureau of Reclamation have varied from about 300 to 700 lb per sq in., depending upon the characteristics of concrete and rock specimens as determined by laboratory tests. The value of the friction coefficient has been taken as 0.65 in all cases, instead of 0.70, as used in Table 9 of Mr. Henny's paper.

The aim of the Bureau of Reclamation has been to keep the minimum shear-friction factor of safety greater than 5 during the most severe condition of reservoir load combined with maximum horizontal and vertical earthquake accelerations. This is easily done in designing gravity dams of ordinary height but requires unusually careful planning in designing dams of unprecedented height, such as Shasta (in California) and Grand Coulee.

Factors of safety against failure by shear along vertical planes may be calculated by methods similar to those used in computing shear-friction factors of safety. In this case the computations for the factor of safety do not include weight effect. They depend only on total shearing forces and total shearing strengths along vertical planes.

Stress Conditions

One of the most important developments in masonry dam design during the fifteen years 1924–1939, if not the most important, has been the gradual im-

provement in methods of analyzing stress conditions, with the accompanying gradual increase in allowable working stresses. This development in technical design procedure has been a natural result of the necessity for building higher and higher structures. Dams such as Boulder, Grand Coulee, and Shasta could not be built if working stresses had to be limited to 10 or 12 tons per sq ft, as they were in designing dams built during the first few years of the twentieth century. In dams of unprecedented height, such as those just mentioned, stresses due to dead loads alone greatly exceed the earlier assumptions of maximum allowable pressures.

Engineers engaged on the design of important masonry dams today recognize that the safety of the structure from the viewpoint of stress conditions should be judged on the basis of the three-dimensional state of stress rather than on the basis of the maximum direct stress in one direction, the condition which usually constituted the governing criterion in early examples of masonry dam design. For instance, at the upstream face of a high curved masonry dam, reservoir pressures near the base, together with horizontal circumferential stresses and Poisson's ratio effects, may offset the calculated vertical tension stresses which might otherwise be considered detrimental. In arch dams, principal stresses along abutment locations may be more pertinent than either horizontal arch stresses or vertical cantilever stresses. In gravity dams built at canyon sites where abutment slopes are relatively steep, stresses caused by twist effects may be important considerations. In unusually high masonry dams, shearing stresses may constitute a more important criterion of safety than direct stresses. Stress concentrations at boundary irregularities, around gallery openings, and at other local discontinuities may also need to be investigated carefully (27).

Local stress concentrations may be studied by theoretical analytical methods, by photoelastic tests, or by actual measurements on small laboratory models. All three methods have been used by the Bureau of Reclamation in connection with design work on masonry dams of both arch and gravity types (28) (29) (30). Whenever two or more methods of investigation were used for the same problem, results by the different methods were found to check satisfactorily.

Formulas for calculating stress conditions in the interior of a masonry dam, based on a straight-line distribution of vertical stress and a parabolic distribution of shear stress, were developed during the design work on Boulder Dam. The derivations were completed about the time that W. H. Holmes, Assoc. M. Am. Soc. C. E., pubished his paper (31). The formulas were accurately derived, using methods which were substantially the same as those subsequently suggested by W. C. Huntington, M. Am. Soc. C. E., in his discussion of Mr. Holmes' paper. It is interesting to note that shearing stresses calculated by using horizontal planes 1 ft, or even 0.1 ft, apart, as originally proposed by the late William Cain (32), M. Am. Soc. C. E., and used by Mr. Holmes, did not agree satisfactorily with those calculated by the theoretical formulas. In order to secure a satisfactorily accurate result by the two-plane method, it was necessary to take horizontal planes 0.001 ft apart. In 1937 an English engineer, Serge Leliavsky, proposed a graphical method (33) of determining shearing

stresses in gravity dams which gives results substantially the same as the theoretical formulas derived by the Bureau of Reclamation.

From the viewpoint of stress conditions, the most noteworthy modern improvements in methods of masonry dam design have been the development of methods of analyzing effects of foundation and abutment deformations and methods of determining the true nonlinear distribution of stress within the structure. The analysis of foundation and abutment deformations was begun by Fredrik Vogt, Assoc. M. Am. Soc. C. E., in 1925 and was further developed by engineers of the Bureau of Reclamation in connection with the work on Owyhee (in Oregon) and Boulder dams (34). Methods of determining the true nonlinear distribution of stress within the dam, including the evaluation of shearing stresses along both horizontal and vertical planes, as well as the evaluation of direct stresses in all directions, were developed by the engineers of the Bureau in connection with the design work on Boulder and Grand Coulee dams, the investigation being conducted by both laboratory and analytical methods. Determinations of nonlinear stress conditions have shown that, although the linear assumption may not be greatly in error in the upper central portions of the dam, it usually is not sufficiently correct for locations near the foundations and abutments. Comparisons of calculated interior stresses based on an assumption of a linear distribution, with nonlinear stresses determined by model measurements, are given in the bulletin on "Slab Analogy Experiments," published in 1938 as one of the final reports on the Boulder Canyon Project (35). Similar comparisons in which nonlinear stresses were determined by trial load methods were published in a subsequent bulletin of the same series (36).

The trial load method of determining nonlinear stresses in the interior of a masonry dam is based on the adjustment of horizontal and vertical beam deflections at all conjugate points in the interior of the dam and in a large foundation block which is included in the analysis as an integral part of the structure. Results obtained by such analyses agree very satisfactorily with those secured by measurements on laboratory models. Although the Bureau's investigations of nonlinear stress conditions were begun in connection with the design of Boulder Dam, a massive arched structure, similar investigations have also been made for straight gravity dams such as Norris (in Tennessee) and Grand Coulee. Drawings showing nonlinear stress conditions determined by measurements made on slab models of Boulder, Norris, and Grand Coulee dams were published in 1934 and 1938 (37) (38).

Summary

The basic assumptions involved in analyzing stresses in masonry dams have been listed in preceding subdivisions of this paper. Summarizing briefly, the more important assumptions are numbers 3, 6, 7, 8, and 10. In other words, the fundamental assumption made in analyzing stresses is that the dam is a homogeneous, uniformly elastic structure, built on a uniformly elastic foundation, and abutting against uniformly elastic canyon walls.

APPENDIX

Bibliography

(1) **"Geology of Boulder and Norris Dam Sites,"** by Charles P. Berkey, M. Am. Soc. C. E., *Civil Engineering*, January, 1935, pp. 24–28.

(2) **"Actual Deflections and Temperatures in a Trial-Load Arch Dam,"** by A. T. Larned and W. S. Merrill, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 99 (1934), pp. 897–961.

(3) **"Special Cements for Mass Concrete,"** by J. L. Savage, M. Am. Soc. C. E., *Bulletin*, 2d Cong., International Commission on Large Dams, World Power Conference, Bureau of Reclamation, 1936.

(4) **"Grouting Contraction Joints at Boulder Dam,"** by James B. Hays, M. Am. Soc. C. E., *Civil Engineering*, February, 1937, pp. 126–130.

(5) **"Bullards Bar Dam, North Fork of Yuba River, California,"** by Culbert W. Faries, *Western Construction News*, April 25, 1927, pp. 39–41.

(6) **"Core Studies in the Hydraulic-Fill Dams of the Miami Conservancy District,"** by Charles H. Paul, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV, 1922, pp. 1181–1236.

(7) **"Puddle Core Investigations at Tieton Dam, Washington,"** by Ivan E. Houk, M. Am. Soc. C. E., *Engineering News-Record*, September 30, 1926, pp. 544–547.

(8) **"Uplift Pressure in Gravity Dams,"** by Ivan E. Houk, *Western Construction News*, July 25, 1930, pp. 344–349.

(9) **"Uplift Pressure in Masonry Dams—Measurements at Existing Structures,"** by Ivan E. Houk, *Civil Engineering*, September, 1932, pp. 578–580.

(10) **"Temperature Control of Mass Concrete in Large Dams,"** by Clarence Rawhouser, Assoc. M. Am. Soc. C. E., *Dams and Control Works*, Bureau of Reclamation, 2d Ed., 1938, pp. 243–251.

(11) **"Flow of Heat in Dams,"** by Robert E. Glover, *Journal*, Am. Concrete Inst., November-December, 1934, pp. 113–124.

(12) **"Temperature Variations in Concrete Dams,"** by Ivan E. Houk, *Western Construction News*, December 10, 1930, pp. 601–608.

(13) **"Setting Heat and Concrete Temperature,"** by Ivan E. Houk, *Western Construction News*, August 10, 1931, pp. 411–415.

(14) **"Design of Gravity Dam in San Gabriel Canyon to Resist Earthquakes,"** by Samuel B. Morris, M. Am. Soc. C. E., and C. E. Pearce, Assoc. M. Am. Soc. C. E., *Bulletin*, Seismological Soc. of Am., September, 1929, pp. 143–155.

(15) **"Water Pressures on Dams During Earthquakes,"** by H. M. Westergaard, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 98 (1933), pp. 418–472.

(16) **"Checking Arch Dam Designs with Models,"** by J. L. Savage and Ivan E. Houk, *Civil Engineering*, May, 1931, pp. 695–699.

(17) **"Model Tests Confirm Design of Hoover Dam,"** by J. L. Savage and Ivan E. Houk, *Engineering News-Record*, April 7, 1932, pp. 494–499.

(18) **"Model of Calderwood Arch Dam,"** by A. V. Karpov and R. L. Templin, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 100 (1935), pp. 185–262.

(19) **"Arch Dam Investigation,"** a report by the Committee, The Engineering Foundation, Vol. I, November, 1927; see also *Proceedings*, Am. Soc. C. E., May, 1928, Part 3.

(20) **"Arch Dam Investigation,"** a report by the Committee, The Engineering Foundation, Vol. II, May, 1934.

(21) **"Arch Dam Investigation,"** a report by the Committee, The Engineering Foundation, Vol. III, May, 1933.

(22) **"Analysis of Arch Dams by the Trial Load Method,"** by C. H. Howell, M. Am. Soc. C. E., and the late A. C. Jaquith, *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 1191–1316.

(23) **"Trial Load Method of Analyzing Arch Dams,"** *Bulletin No. 1*, Part V—Technical Investigations, Boulder Canyon Project Final Reports, Bureau of Reclamation, 1938.

(24) **"Trial Load Analyses of Curved Concrete Dams,"** by Ivan E. Houk, *The Engineer*, July 5 and 12, 1935, pp. 382–384 and 414–415.

(25) **"Twist Effects in Straight Gravity Dams,"** by Ivan E. Houk, *The Engineer*, December 17 and 24, 1937, pp. 676–678 and 702–705.

(26) **"Stability of Straight Concrete Gravity Dams,"** by the late D. C. Henny, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 99 (1934), pp. 1041–1123.

(27) **"Conformity of Design Assumptions and Actual Conditions in Masonry Dams,"** by A. V. Karpov, *The Military Engineer*, November-December 1938, pp. 418–421.

(28) **"The Stress Function and Photo-Elasticity Applied to Dams,"** by John H. A. Brahtz, *Transactions*, Am. Soc. C. E., Vol. 101 (1936), pp. 1240–1301.

(29) **"Stresses Around Circular Holes in Dams and Buttresses,"** by I. K. Silverman, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 103 (1938), pp. 133–180.

(30) **"Stresses Around Galleries in Concrete Dams,"** by Ivan E. Houk, *The Engineer*, October 7, 1938, pp. 382–385.

(31) **"Determination of Principal Stresses in Buttresses and Gravity Dams,"** by W. H. Holmes, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. C E., January, 1932, p. 71; later in *Transactions*, Am. Soc. C. E., Vol. 98 (1933), pp. 971–1038.

(32) **"Stresses in Masonry Dams,"** by the late William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXIV, September, 1909, pp. 208–230.

(33) **"Shearing Stresses in Gravity Dams,"** by Serge Leliavsky, *Journal*, Inst. of Civ. Engrs., No. 4, 1937–1938, pp. 73–96.

(34) **"Ueber die Berechnung der Fundamentdeformation,"** by Fredrik Vogt, Assoc. M. Am. Soc. C. E., *Det Norske Videnskaps Akademi*, Oslo, Norway, 1925; see also Chapter III of reference (23).

(35) **"Slab Analogy Experiments,"** *Bulletin No. 2*, Part V—Technical Investigations, Boulder Canyon Project Final Reports, Bureau of Reclamation, 1938.

(36) **"Stress Studies for Boulder Dam,"** *Bulletin No. 4*, Part V—Technical Investigations, Boulder Canyon Project Final Reports, Bureau of Reclamation, 1939.

(37) **"Dam Stresses and Strains Studied by Slice Models,"** by J. L. Savage, *Engineering News-Record*, December 6, 1934, pp. 720–723.

(38) **"Structural Model Testing of Dams,"** by Eldred D. Smith, *Dams and Control Works*, Bureau of Reclamation, 2d Ed., 1938, pp. 213–226.

DESIGN OF ARCH DAMS

By R. S. Lieurance,[4] Esq.

Synopsis

The purpose of this paper is to present a comprehensive series of tables to facilitate the computation of forces, moments, and radial deflections in the design of arch dams. Seven basic load conditions are thus provided for, in an Appendix, and the text comprises a brief discussion of design problems involved.

General

An ideal arch dam may be defined as a structure by which the total water load is transmitted to the abutments by arch action. The abutments may be either the natural walls of a canyon or they may be constructed artificially. This ideal condition is seldom realized, and a part of the water load is usually carried by the gravity elements of the structure.

The gravity elements and vertical beam action introduce a complexity in the design, and a true or nearly true arch dam is sometimes produced by the introduction of an articulated joint at the base of the dam. This may be permissible in exceptional cases in which it is impossible to design for the high shears found at the base, and if it is possible to develop an articulated joint that is 100% efficient throughout its entire length. The failure of this joint to operate at any point will introduce a discontinuity in the structure with consequent secondary stresses of undetermined magnitude. It is the opinion of the writer that even if the ideal conditions can be met for the articulated structure, it is not desirable to destroy any part of the structural strength of the dam in order to simplify its analysis.

The ideal site for an arch dam is a canyon having a small ratio of length to height and having reasonably sound abutments which are so nearly symmetrical that symmetry can be produced with a small amount of excavation.

An economical and relatively simple structure will result when the ratio of length to height does not exceed about $2\frac{1}{2}$ to 1. It will be found that a considerable proportion of the water load is carried by gravity action when this ratio reaches 5 to 1. At about that value, the arch will become quite massive, and its additional length and high unit costs make its selection doubtful unless it is possible to take advantage of topographic conditions in the foundation, as in the case where the narrow part of the canyon has been scoured to a greater depth than the section upstream. In that case it may be economical to take advantage of this condition with the curved structure. It should be noted that the ratios given are relative only, and that local conditions will alter them considerably.

[4] Construction Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Altus, Okla.

The effect of lack of symmetry or lack of continuity cannot be predicted, but must be determined by analysis. These effects may be so great as to preclude the use of an arch dam, unless it is found feasible to construct an abutment or thrust block which produces the required symmetry.

These conditions affect only the economy of the structure, as it is possible to design for any reasonable condition, but the work of design becomes much more difficult, and should not be attempted until the designer has had considerable experience in the design of arch dams.

The conditions that must be met at the abutments are complex, and the most severe loading conditions may not occur at the time of maximum stress in the dam, which usually occurs with maximum water load and minimum temperature of the concrete. At this time, the resultant force applied to the abutment will have its most dangerous direction; but usually it will have its minimum magnitude in the top of the abutment.

This condition of minimum magnitude may not hold in special cases. There are cases in which the water load is transferred by tangential shear toward the upper part of the abutment. This may be caused by unusual topographic conditions, or by the necessity of building the upper portions of the dam to a greater than normal thickness to provide for roadway or ice load.

It follows that the maximum abutment load will occur in normal cases with full water load and maximum concrete temperatures. This is not the most critical case for natural abutments, as the shear produced by the plus temperature in the arches is toward the upstream face at the abutment, which throws the resultant farther back into the hill. This condition must be considered when artificial abutments are necessary, as the lateral stability of the abutment must be considered.

The magnitude and variation in direction of the resultant forces must be computed, and sufficient mass must be provided in the abutment to take the most severe conditions with a high factor of safety. Due to the usual jointing in the rock and possible presence of hydrostatic pressure in the abutment, and the usual presence of tension in the abutment at the intersection of the upstream face and abutment, it is advisable to consider as acting only that part of the abutment which comes under direct bearing from the arch.

It should be noted that the angle of intersection of the arch tangent and the abutment is not a criterion for judging the direction of the applied force. The arch thrust is only one component of the true force, and is separated for convenience in computation. The arch shear is the other force component. There are many cases in which the resultant abutment force is directed out of the abutment toward the river. This will occur with thick arches having a small central angle and a minimum temperature condition. It will also occur with thin arches having a central angle close to 180°.

Analysis

The structure to be analyzed is inherently complex, as is the distribution of forces throughout the structure. This distribution is most clearly defined by the methods of the theory of elasticity, but as there is no known formal method of solution, the effects of the various loads must be evaluated by the methods

of ordinary strength of materials; and the final solution must be approached by trial.

The general conditions imposed by the theory of elasticity are that each element in the dam must be in static equilibrium, and that the continuity of the structure is undisturbed by loading. It also follows that the state of stress throughout the structure is determined by the state of stress on any mutually perpendicular coordinate system. The obvious system to follow is a modified system of cylindrical coordinates, in which vertical planes will cut radial slices through the dam, called cantilevers, and horizontal planes will cut horizontal arches. This method must be considered as the one giving the greatest ease of solution, as the sections cut from the dam by planes spaced at unit distance have the characteristics of members that can be treated by the ordinary mechanics of materials. This arbitrary selection of a coordinate system in no way affects the distribution of load within the structure, but gives a means of evaluating vertical and horizontal components of stress and strain; and, as the problem is determinate from these two sets of stresses, the true distribution of load and method by which it is carried to the abutments can be evaluated.

The validity of the method has been demonstrated by the remarkable accuracy with which field and laboratory measurements have been checked by analysis.

The trial-load method of analyzing dams, in its present state of development, is the result of many years of study and research at the U. S. Bureau of Reclamation, having as its objective the development of a method that would be (1) workable and (2) capable of meeting the rigorous test of laboratory and field measurements. The earlier works in this field, notably that of H. E. Gumer, Professor Rohn, and Albert Stukie in Europe, and the work of Julian Hinds and C. H. Howell, Members, Am. Soc. C. E., and the late A. C. Jaquith, in the United States, have paved the way for the present methods.

Later developments in the method have been made possible by contributions from Fredrik Vogt, Assoc. M. Am. Soc. C. E., H. M. Westergaard and Ivan E. Houk, Members, Am. Soc. C. E. To Robert E. Glover, engineer, Bureau of Reclamation, goes the credit of having made the first trial-load analysis that includes the effect of torsion, tangential shear, and Poisson's ratio.[5] The method as presented by Mr. Glover represents the greatest refinement found practicable in arch dam analysis.

Choice of Section for Dam

The choice of the section for a given dam site depends entirely on its individual characteristics. The most suitable section may not be found without careful study and alternate designs which are selected for the purpose of evaluating these characteristics. Certain fundamentals will be of assistance in making the layout.

The economical central angle of an arch is one regarding which there are many misconceptions. Publications often give this angle as 120°, a value that results from the thin cylinder formula without regard for temperature. The

[5] *Bulletin No. 1*, Part V, Technical Investigations, Boulder Canyon Final Reports, Bureau of Reclamation, U. S. Dept. of the Interior.

elastic theory gives a value of approximately 150° to 160°; and special cases in which high temperature stresses are used give values close to 180° for the economical central angle.

It should be noted that the most economical central angle can be obtained only for one set of working conditions; and the most economical angle seldom can be used, due to the abutment and foundation conditions, as the angle of the resultant at the abutment may be of greater importance.

The zone of higher stress is likely to occur at about two thirds of the height of the structure, at which point most of the load is carried by arch action. The examination of a given section at this point, using the full water load, may disclose a weakness or overdesign in the structure and eliminate the necessity of making a detailed analysis of the entire dam.

In order that a suitable section may be more readily arrived at by a small organization lacking the time and personnel to make extensive preliminary design studies there is presented herein a tabulation of constants, the use of which will eliminate a considerable number of the calculations necessary to complete a radial adjustment. The only calculations necessary to obtain all the arch data required for the radial adjustment can be made with a slide rule in a short time. The value of such data is best illustrated by quoting from the Reclamation Bureau bulletin on "The Trial Load Method of Analyzing Arch Dams":[5]

> "The radial adjustment constitutes the most important step in the trial load analysis. Although tangential and twist adjustments are usually advisable, they are of secondary importance when compared with the radial adjustment. The reason for this is evident from the fact that radial movements are much larger than tangential or angular movements. An analysis based on the adjustment of radial deflections only is often made as a preliminary study. Such an analysis has been found to give a fair indication of maximum stresses. This type of analysis is used to compare preliminary designs for a dam, the complete analysis being used to determine stresses in the adopted design."

Tabular Coefficients and Formulas for Designing Circular Arches

The Appendix includes fourteen tables for use in designing circular arches, two being necessary for each load. For any load Table (*a*), of a pair, lists the factors necessary to obtain the forces and moments at the crown and the abutment of the arch, whereas Table (*b*) of that pair lists the factors necessary to obtain the deflections of the quarter points of the arch. For the most part the values are to four significant figures; however, in some cases only three significant figures are used. Some of the tabular values have several zeros following the decimal point. In such cases the number of zeros between the decimal point and the first significant figure is designated by a superior number; for example, 0.0000154 is written 0.0^4154.

The unit loads listed are as follows: No. 1, a uniform radial load (Table 3, Appendix); Nos. 2, 3, 4, and 5, triangular radial loads (Tables 4, 5, 6, and 7, Appendix); temperature load, consisting of a unit temperature change of minus 10° (Table 8, Appendix); and concentrated radial load, applied at the abutment (Table 9, Appendix). By means of these loads it is possible to produce any load that varies as a straight line between quarter points. Each load is illustrated on its respective sheet in the tables.

The following definitions as to signs are used throughout the tables:

1. A positive thrust (H) produces compression;
2. A positive moment (M) tends to produce compression in the extrados of the arch;
3. A positive shear (V) tends to move a section of the arch upstream with respect to a section nearer the abutment;
4. A positive radial load acts downstream toward the center of the arch;
5. A positive deflection is a movement upstream; and
6. A temperature rise is assumed positive.

Inasmuch as the values listed in Tables 3 to 9 (Appendix) include the effects of bending, ribshortening, shear, and abutment deformations, certain assumptions over the ordinary were necessary in their calculations. These are as follows:

The modulus of elasticity of the concrete in tension and compression (E_c) is equal to the modulus of elasticity of the foundation and abutment rock (E_R). This assumption is valid for most cases. If conditions are encountered which differ greatly from this, independent calculation of the arch data will be necessary using the constants

$$\alpha = \frac{5.075}{E_R\, t^2} \qquad (1a)$$

$$\beta = \frac{1.556}{E_R} \qquad (1b)$$

and

$$\gamma = \frac{1.785}{E_R} \qquad (1c)$$

in which: t = thickness of the arch ring; α = average rotation in a plane normal to the foundation surface due to the unit moment load; β = average deformation normal to the foundation surface due to the normal unit load; and γ = average deformation in the plane of the foundation surface due to unit shear load. In an adjustment using Tables 3 to 9 (Appendix) the factors expressed by Eqs. 1 must be used also in computing the cantilever deflections.

Acknowledgments

Acknowledgment is made to assistant engineer C. F. Palmetier for his work in computation and arrangement of the tables; to assistant engineer Dwight S. Young for assembly and checking; to assistant engineer Raymond G. Rolin, Jun. Am. Soc. C. E., for his work in checking the theoretical work; and to Mr. Houk for his review of the paper.

This work has been prepared under the supervision of J. L. Savage, M. Am. Soc. C. E., chief designing engineer. All construction and engineering of the Department of the Interior, Bureau of Reclamation, is under the general supervision of R. F. Walter, M. Am. Soc. C. E., chief engineer with headquarters in Denver, Colo., and all activities of the Bureau are under the general charge of

John C. Page, M. Am. Soc. C. E., commissioner, with headquarters in Washington, D. C.

APPENDIX

The Use of the Tables

The caption of each table describes the load and the type of values to be found on that sheet (forces and moments, or deflections). In any table the extreme left column is designated ϕ_A and represents the half central angle of the arch. At the top of the tables are listed the values $\frac{t}{r}$, in which t is the arch thickness and r is the radius of the arch center line. Straight-line interpolation between the values $\frac{t}{r}$ and ϕ_A to obtain the desired value from the table will be sufficiently accurate for most cases.

For example, consider a 63° arch with a radius of 500 ft and a thickness of 40 ft: E_c = 4,000,000 lb per sq in. = 576,000,000 lb per sq ft; and $\frac{t}{r} = 0.080$. The thrusts at the crown and the quarter points, due to load No. 1, are interpolated as shown in Table 1. From Table 1(*a*), h = 1,024; and $H = 1{,}024 \times 500 = 512{,}000$ lb. From Table 1(*b*), $K_1 = -23{,}210$; and

$$\Delta = \frac{-23{,}210 \times 500}{576{,}000{,}000} = -0.02035.$$

TABLE 1.—Use of Tables 3 to 9; Illustrative Example

Values of ϕ_A, in degrees	Values of $\frac{t}{r}$: 0.075	0.080	0.100	0.075	0.080	0.100
	(*a*) Thrust at the Crown Due to Load No. 1			(*b*) Deflection at the Quarter Point Due to Load No. 1		
60	+1,022	+1,022	+1,022	−24,560	−23,440	−18,950
63	......	+1,024	......	......	−23,210	......
70	+1,030	+1,031	+1,035	−24,470	−23,360	−18,900

The variety of sections which are possible for cantilevers makes tabulations such as those presented for arches impossible.[6]

Radial Load Description.—Referring to Fig. 1, P is the total load at the abutment—in pounds per square feet when the load is distributed and in pounds when the load is concentrated. The loads indicated by the diagrams in Tables

[6] To compute cantilever stresses and deflections, refer to *Bulletin No. 1*, Part V, Technical Investigations, Boulder Canyon Final Reports, Bureau of Reclamation, U. S. Dept. of the Interior, Chapter IV, p. 63.

3 to 9 are unit radial loads applied at the upstream face (except the unit temperature load) and may be combined into any pattern that varies as a straight line between quarter points. The various loads are described in Table 2. The

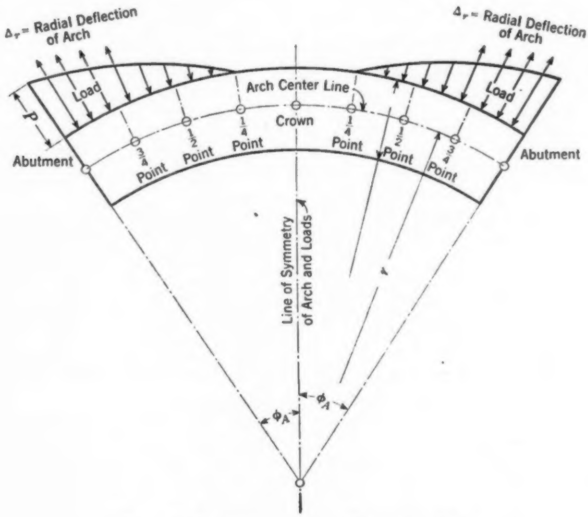


FIG. 1.—PROPERTIES OF A CIRCULAR ARCH

unit load shown in the diagrams in Tables 3 to 9 (Appendix) results in forces or moments determined by the corresponding formulas given in the tables.

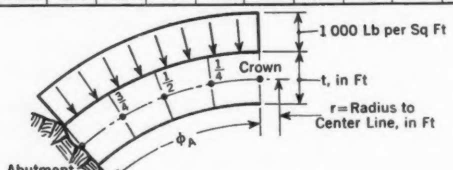
TABLE 2.—DESCRIPTION OF RADIAL LOADS

Table No.	Load No.	Intensity, in pounds per square foot	Pattern	Variation; maximum at abutment to zero at:
3	1	10,000	Uniform	Uniform
4	2	10,000	Triangular	Three-quarter point
5	3	10,000	Triangular	Half point
6	4	10,000	Triangular	Quarter point
7	5	10,000	Triangular	Crown
8	Temperature	Uniform change of −10° F		
9	Concentrated	1,000 lb, applied radially at the abutment		

Positive Sign Conventions.—Fig. 2 shows the direction of positive forces and moments, and the direction of forces and moments due to positive loads. Loads directed toward the arch center, or downstream, are positive. Deflections away from the arch center, or upstream, are positive. Thrusts are positive when they tend to produce compression; moments when they tend to produce compression in the extrados; and shears when they tend to move a section upstream with respect to a section nearer the abutment. The temperature table (Table 8, Appendix) is based on a drop in temperature.

TABLE 3(*a*).—Forces and Moments for Radial Load No. 1

ϕ_A	Values of t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	Values of h at Crown															
10	310.1	117.5	67.91	48.84	39.69	27.65	26.22	26.46	27.20	28.14	29.18	30.28	31.41	32.56	33.73	34.91
20	880.6	656.0	477.5	360.7	286.2	153.4	124.8	116.8	115.4	116.6	119.2	122.5	126.2	130.2	134.4	138.7
30	983.1	919.3	831.9	740.8	657.4	407.6	318.0	283.7	270.7	267.4	269.1	273.6	279.8	287.1	295.1	303.7
40	1003	988.7	960.8	923.8	876.4	683.7	565.3	505.7	477.7	466.7	465.4	469.8	477.8	488.3	500.4	513.6
50	1009	1010	1005	993.8	978.9	877.1	786.6	728.3	696.4	682.4	679.9	685.6	696.3	710.6	727.4	746.1
60	1011	1018	1022	1022	1021	987.2	942.3	907.2	886.8	879.5	882.5	893.0	909.0	929.1	952.1	977.5
70	1012	1021	1029	1035	1039	1046	1039	1032	1031	1038	1056	1072	1096	1125	1156	1189
80	1012	1023	1033	1041	1049	1079	1097	1114	1132	1155	1182	1213	1249	1287	1328	1371
90	1012	1024	1035	1045	1055	1097	1132	1166	1200	1237	1277	1320	1366	1415	1466	1519
	Values of m at Crown															
10	3.967	5.593	6.424	7.052	7.592	9.753	11.48	12.99	14.35	15.62	16.83	17.98	19.10	20.19	21.26	22.31
20	2.817	8.293	13.18	16.90	19.77	28.68	34.77	40.02	44.86	49.44	53.83	58.08	62.23	66.30	70.29	74.23
30	1.377	5.128	10.31	15.99	21.56	43.05	57.16	68.27	78.03	87.07	95.67	104.0	112.1	120.1	127.9	135.6
40	0.775	3.042	6.593	11.12	16.30	43.96	67.14	85.76	101.6	115.9	129.2	141.8	154.0	165.9	177.6	189.1
50	0.486	1.935	4.295	7.470	11.34	36.58	63.47	87.66	109.9	128.0	145.6	162.0	177.7	192.8	207.5	222.0
60	0.328	1.310	2.928	5.150	7.926	27.98	52.98	78.15	101.7	123.4	143.4	162.1	179.8	196.7	213.1	228.9
70	0.232	0.926	2.075	3.661	5.664	20.84	41.48	63.99	86.29	107.5	126.0	146.0	163.5	180.1	196.0	211.2
80	0.170	0.675	1.512	2.668	4.131	15.42	31.42	49.67	68.39	86.58	103.8	119.9	134.9	148.9	162.1	174.6
90	0.127	0.503	1.122	1.977	3.056	11.39	23.31	37.07	51.29	65.09	77.97	89.73	100.3	109.9	118.4	126.0
	Values of h at Abutment															
10	320.8	131.3	82.64	64.06	55.23	44.32	43.86	45.05	46.73	48.60	50.58	52.61	54.67	56.76	58.85	60.96
20	888.6	678.3	511.2	402.3	333.0	212.0	188.9	185.2	187.6	192.5	198.7	205.5	212.8	220.3	228.0	235.9
30	987.0	933.5	859.5	782.3	711.7	503.7	434.5	413.1	410.3	415.8	425.6	437.9	451.7	466.3	481.7	497.4
40	1005	997.2	978.8	953.3	919.9	786.9	710.9	679.9	673.0	679.2	692.8	710.8	731.6	754.2	778.1	802.9
50	1010	1015	1016	1014	1009	965.7	929.8	914.6	916.5	929.8	950.6	976.5	1006	1037	1070	1105
60	1012	1021	1029	1036	1042	1056	1065	1079	1100	1127	1160	1196	1236	1277	1320	1364
70	1012	1024	1035	1045	1055	1098	1137	1176	1216	1260	1307	1353	1403	1454	1506	1558
80	1012	1025	1037	1049	1060	1117	1172	1226	1281	1337	1393	1450	1508	1566	1625	1684
90	1013	1025	1038	1050	1063	1125	1188	1250	1313	1375	1438	1500	1563	1625	1688	1750
	Values of m at Abutment															
10	-6.704	-8.194	-8.306	-8.158	-7.947	-6.918	-6.159	-5.601	-5.175	-4.839	-4.569	-4.346	-4.160	-4.001	-3.865	-3.746
20	-5.136	-13.96	-20.60	-24.67	-27.05	-29.92	-29.32	-28.31	-27.33	-26.45	-25.68	-24.99	-24.39	-23.85	-23.38	-22.95
30	-2.563	-9.032	-17.23	-25.43	-32.71	-53.06	-59.33	-61.20	-61.55	-61.32	-60.86	-60.31	-59.74	-59.19	-58.65	-58.14
40	-1.447	-5.448	-11.34	-18.41	-27.24	-59.29	-78.42	-88.36	-93.66	-96.60	-98.28	-99.24	-99.78	-100.1	-100.2	-100.2
50	-0.903	-3.476	-7.468	-12.59	-18.54	-51.96	-79.74	-98.71	-110.1	-119.4	-125.0	-128.9	-131.8	-133.8	-135.4	-136.6
60	-0.602	-2.339	-5.089	-8.715	-13.07	-40.94	-69.64	-93.27	-111.2	-124.4	-133.1	-141.4	-146.9	-151.2	-154.6	-157.3
70	-0.419	-1.633	-3.574	-6.166	-9.329	-30.93	-56.01	-79.28	-98.75	-114.3	-125.0	-135.9	-143.3	-149.2	-153.8	-157.6
80	-0.299	-1.169	-2.564	-4.436	-6.738	-22.95	-43.00	-62.90	-80.60	-95.42	-107.4	-116.9	-124.4	-130.3	-134.9	-138.5
90	-0.217	-0.849	-1.863	-3.224	-4.900	-16.83	-31.96	-47.39	-61.42	-73.30	-82.87	-90.29	-95.87	-99.92	-102.7	-104.6
	Values of v at Abutment															
10	-122.0	-157.6	-168.4	-173.8	-177.6	-190.6	-201.7	-212.5	-223.2	-233.9	-244.6	-255.2	-265.9	-276.5	-287.2	-297.8
20	-45.11	-126.2	-191.5	-235.8	-265.5	-332.3	-363.5	-387.6	-409.4	-430.4	-450.9	-471.1	-491.2	-511.3	-531.2	-551.1
30	-14.70	-52.85	-102.8	-154.6	-202.6	-358.7	-434.7	-483.2	-520.9	-553.8	-584.2	-613.2	-641.3	-669.0	-696.2	-723.2
40	-6.104	-23.33	-49.28	-81.14	-119.6	-283.7	-399.9	-478.4	-536.6	-583.8	-624.9	-662.2	-697.2	-730.7	-763.1	-794.7
50	-2.978	-11.60	-25.23	-43.01	-64.05	-189.9	-307.1	-399.7	-472.0	-530.6	-580.3	-623.9	-663.6	-700.5	-735.5	-769.1
60	-1.612	-6.319	-13.89	-24.01	-36.37	-119.4	-212.4	-296.9	-368.7	-429.1	-480.7	-525.7	-565.9	-602.7	-636.8	-669.0
70	-0.929	-3.655	-8.067	-14.03	-21.41	-73.93	-139.2	-204.6	-264.3	-316.7	-358.5	-402.6	-438.2	-470.2	-499.6	-526.7
80	-0.559	-2.198	-4.858	-8.466	-12.95	-45.73	-88.69	-134.2	-177.6	-216.9	-251.7	-282.2	-309.0	-332.8	-354.0	-373.1
90	-0.344	-1.352	-2.985	-5.201	-7.956	-28.22	-55.26	-84.45	-112.7	-138.4	-160.8	-180.0	-196.2	-209.8	-221.1	-230.6



Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$
$M = mr^2$
$V = vr$

DIAGRAM OF RADIAL LOAD NO. 1

TABLE 3(*b*).—Radial Deflections for Radial Load No. 1

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF K_1 AT CROWN																
10	-27190	-5956	-2626	-1604	-1168	-628.7	-538.2	-511.8	-507.1	-511.5	-520.6	-532.5	-546.2	-561.1	-576.8	593.2
20	-71560	-28760	-15020	-9141	-6208	-2256	-1557	-1327	-1232	-1193	-1180	-1183	-1194	-1212	-1233	-1258
30	-77910	-38350	-28330	-17070	-12720	-4936	-3118	-2462	-2169	-2022	-1949	-1914	-1905	-1910	-1927	-1949
40	-78590	-40260	-27090	-20280	-16070	-7428	-4804	-3716	-3186	-2903	-2745	-2657	-2612	-2595	-2597	-2612
50	-78550	-40560	-27720	-21200	-17210	-8900	-6085	-4779	-4061	-3695	-3463	-3324	-3244	-3202	-3187	-3191
60	-78510	-40540	-27820	-21400	-17530	-9560	-6823	-5485	-4737	-4293	-4017	-3846	-3741	-3682	-3654	-3648
70	-78460	-40490	-27780	-21400	-17560	-9798	-7168	-5869	-5126	-4671	-4374	-4199	-4082	-4015	-3980	-3969
80	-78540	-40470	-27730	-21350	-17520	-9849	-7289	-6033	-5313	-4867	-4582	-4397	-4280	-4209	-4161	-4160
90	-78730	-40480	-27700	-21310	-17470	-9821	-7299	-6071	-5369	-4935	-4657	-4476	-4360	-4290	-4254	-4241
VALUES OF K_1 AT $\frac{1}{4}$ POINT																
10	-24270	-5403	-2418	-1496	-1100	-608.6	-525.5	-503.2	-500.2	-505.5	-515.3	-527.7	-541.7	-556.8	-572.9	-589.3
20	-63370	-25680	-13520	-8290	-5680	-2124	-1490	-1283	-1199	-1165	-1156	-1160	-1173	-1192	-1215	-1240
30	-68780	-34040	-21710	-15310	-11470	-4558	-2930	-2341	-2041	-1949	-1885	-1858	-1853	-1862	-1874	-1906
40	-69230	-35610	-24060	-18080	-14390	-6777	-4452	-3485	-3015	-2764	-2626	-2552	-2516	-2505	-2511	-2530
50	-69080	-35790	-24550	-18830	-15330	-8054	-5581	-4431	-3806	-3479	-3278	-3159	-3092	-3060	-3052	-3061
60	-68960	-35700	-24560	-18950	-15560	-8602	-6210	-5042	-4332	-4002	-3765	-3619	-3532	-3485	-3466	-3466
70	-68800	-35580	-24470	-18900	-15550	-8777	-6485	-5356	-4712	-4319	-4062	-3916	-3826	-3765	-3740	-3737
80	-68740	-35470	-24370	-18810	-15470	-8790	-6565	-5476	-4853	-4469	-4225	-4069	-3972	-3915	-3887	-3883
90	-68730	-35400	-24280	-18720	-15390	-8738	-6548	-5486	-4880	-4507	-4268	-4115	-4019	-3962	-3934	-3928
VALUES OF K_1 AT $\frac{1}{2}$ POINT																
10	-16520	-3903	-1843	-1193	-908.9	-550.2	-490.8	-477.7	-479.4	-487.6	-499.3	-513.1	-528.2	-544.1	-560.6	-577.7
20	-41800	-17460	-9474	-5982	-4214	-1749	-1299	-1153	-1099	-1081	-1082	-1094	-1112	-1135	-1160	-1188
30	-44840	-22660	-14750	-10610	-8106	-3513	-2401	-1996	-1819	-1737	-1702	-1694	-1702	-1720	-1746	-1777
40	-44800	-23420	-16070	-12270	-9906	-5002	-3474	-2835	-2526	-2366	-2284	-2246	-2235	-2243	-2261	-2290
50	-44420	-23330	-16210	-12590	-10380	-5774	-4202	-3466	-3072	-2868	-2749	-2685	-2657	-2651	-2663	-2685
60	-44100	-23100	-16080	-12540	-10410	-6040	-4548	-3823	-3422	-3190	-3054	-2976	-2936	-2922	-2927	-2945
70	-43730	-22860	-15890	-12400	-10300	-6067	-4651	-3962	-3575	-3344	-3195	-3123	-3081	-3060	-3061	-3076
80	-43420	-22640	-15710	-12240	-10150	-6001	-4633	-3972	-3601	-3378	-3241	-3159	-3113	-3092	-3085	-3101
90	-43110	-22430	-15530	-12080	-10030	-5905	-4563	-3919	-3557	-3340	-3205	-3122	-3074	-3050	-3044	-3051
VALUES OF K_1 AT $\frac{3}{4}$ POINT																
10	-6869	-1931	-1054	-760.9	-628.8	-458.0	-434.8	-435.9	-445.2	-458.1	-473.0	-489.0	-505.7	-522.9	-540.5	-558.4
20	-15790	-7256	-4305	-2954	-2243	-1201	-1005	-950.3	-939.1	-946.6	-963.7	-968.3	-1012	-1041	-1071	-1101
30	-16350	-8833	-6122	-4676	-3750	-2064	-1629	-1476	-1420	-1406	-1414	-1433	-1461	-1494	-1530	-1568
40	-16000	-8820	-6359	-5084	-4292	-2630	-2104	-1890	-1798	-1763	-1759	-1773	-1798	-1830	-1868	-1910
50	-15630	-8586	-6218	-5022	-4296	-2810	-2323	-2109	-2006	-1968	-1959	-1969	-1992	-2023	-2060	-2102
60	-15330	-8352	-6029	-4865	-4169	-2782	-2344	-2153	-2063	-2025	-2016	-2026	-2047	-2077	-2113	-2153
70	-15020	-8142	-5851	-4708	-4026	-2684	-2260	-2095	-2012	-1978	-1960	-1978	-1996	-2023	-2055	-2091
80	-14740	-7952	-5694	-4566	-3893	-2576	-2171	-1997	-1915	-1878	-1866	-1870	-1883	-1904	-1928	-1958
90	-14460	-7778	-5552	-4442	-3778	-2476	-2072	-1895	-1807	-1764	-1734	-1740	-1745	-1757	-1773	-1792
VALUES OF K_1 AT ABUTMENT																
10	-217.7	-281.3	-300.5	-310.3	-317.0	-340.1	-360.0	-379.3	-398.4	-417.5	-436.5	-455.6	-474.6	-493.6	-512.6	-531.6
20	-80.51	-225.3	-341.9	-420.8	-473.9	-593.2	-648.8	-691.8	-730.9	-768.3	-804.8	-841.0	-876.9	-912.6	-948.2	-983.7
30	-26.25	-94.33	-183.5	-275.9	-361.6	-640.2	-776.0	-862.5	-929.8	-988.5	-1043	-1095	-1145	-1194	-1243	-1291
40	-10.90	-41.64	-87.97	-144.8	-207.5	-506.4	-713.9	-853.9	-957.8	-1042	-1115	-1182	-1245	-1304	-1362	-1419
50	-5.316	-20.71	-45.03	-76.77	-114.3	-338.9	-548.2	-713.4	-842.5	-947.1	-1036	-1114	-1184	-1250	-1313	-1373
60	-2.877	-11.28	-24.79	-42.87	-64.92	-213.1	-379.1	-530.0	-658.1	-765.9	-858.0	-938.4	-1010	-1076	-1137	-1194
70	-1.658	-6.524	-14.40	-25.05	-38.22	-132.0	-248.5	-365.2	-471.7	-565.3	-639.9	-718.6	-782.1	-839.4	-802.5	-940.2
80	-0.9976	-3.923	-8.672	-15.11	-23.12	-81.63	-158.3	-239.5	-317.0	-387.2	-449.3	-503.8	-551.6	-594.0	-631.8	-666.0
90	-0.6140	-2.413	-5.328	-9.284	-14.20	-50.38	-98.64	-150.7	-201.2	-247.0	-287.1	-321.3	-350.2	-374.4	-394.7	-411.6

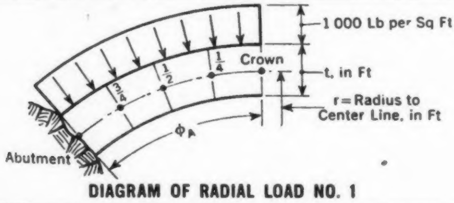


DIAGRAM OF RADIAL LOAD NO. 1

Radial Deflection, in Ft

$$\Delta_r = \frac{K_1 r}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, in Lb per Sq Ft

TABLE 4(*a*).—Forces and Moments for Radial Load No. 2

ϕ_A	Values of t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF h AT CROWN																
10	6.046	3.198	2.885	2.873	2.949	3.477	3.924	4.302	4.641	4.954	5.252	5.541	5.822	6.097	6.369	6.637
20	10.28	10.29	9.976	9.850	9.921	11.42	13.14	14.71	16.13	17.46	18.71	19.92	21.08	22.21	23.32	24.43
30	10.44	11.94	13.14	14.18	15.16	19.68	23.75	27.38	30.67	33.70	36.56	39.29	41.92	44.46	46.95	49.38
40	10.22	11.75	13.25	14.74	16.25	23.99	31.33	37.93	43.88	49.35	54.45	59.29	63.91	68.38	72.72	76.95
50	10.08	11.46	12.85	14.30	15.78	24.53	34.15	43.60	52.48	60.77	68.54	75.94	82.98	89.76	96.33	102.7
60	10.05	11.27	12.52	13.82	15.21	23.51	33.70	44.72	55.81	66.60	76.97	86.93	96.51	105.8	114.7	123.5
70	10.09	11.19	12.32	13.49	14.74	22.24	31.95	43.23	55.33	67.68	79.98	92.06	103.9	115.4	126.6	137.6
80	10.19	11.20	12.23	13.30	14.43	21.16	30.01	40.75	52.85	65.76	79.08	92.52	105.9	119.2	132.3	145.3
90	10.35	11.28	12.24	13.23	14.27	20.34	28.30	38.18	49.69	62.39	75.90	89.90	104.2	118.6	133.0	147.3
VALUES OF m AT CROWN																
10	-.0152	.0336	.0489	.0615	0.727	0.118	0.153	0.183	0.209	0.234	0.256	0.278	0.298	0.318	0.338	0.357
20	-0.104	-.0803	-.0496	-.0233	$-.0^{2}21$	.0633	0.105	0.141	0.172	0.202	0.230	0.257	0.283	0.308	0.333	0.483
30	-0.255	-0.290	-0.315	-0.336	-0.356	-0.475	-0.611	-0.749	-0.885	-1.017	-1.145	-1.271	-1.395	-1.516	-1.636	-1.754
40	-0.441	-0.517	-0.596	-0.681	-0.773	-1.330	-1.960	-2.594	-3.214	-3.820	-4.413	-4.995	-5.569	-6.136	-6.697	-7.253
50	-0.669	-0.776	-0.892	-1.022	-1.169	-2.162	-3.486	-4.897	-6.365	-7.832	-9.284	-10.73	-12.16	-13.57	-14.98	-16.38
60	-0.942	-1.075	-1.221	-1.384	-1.569	-2.870	-4.767	-7.036	-9.576	-12.19	-14.86	-17.54	-20.23	-22.92	-25.61	-28.30
70	-1.259	-1.418	-1.590	-1.781	-1.994	-3.502	-5.813	-8.818	-12.32	-16.14	-20.16	-24.32	-28.55	-32.83	-37.15	-41.50
80	-1.620	-1.804	-2.001	-2.217	-2.468	-4.106	-6.677	-10.17	-14.44	-19.30	-24.59	-30.21	-36.04	-42.04	-48.17	-54.39
90	-2.024	-2.232	-2.454	-2.693	-2.955	-4.714	-7.436	-11.22	-15.99	-21.62	-27.94	-34.80	-42.09	-49.71	-57.59	-65.68
VALUES OF h AT ABUTMENT																
10	6.275	3.475	3.170	3.163	3.241	3.782	4.241	4.633	4.987	5.315	5.629	5.932	6.229	6.520	6.807	7.091
20	10.94	10.97	10.69	10.59	10.67	12.16	13.86	15.41	16.83	18.15	19.41	20.62	21.79	22.93	24.06	25.17
30	11.93	13.26	14.34	15.28	16.16	20.25	23.96	27.28	30.30	33.11	35.77	38.31	40.76	43.14	45.47	47.76
40	12.96	14.20	15.41	16.61	17.83	24.08	30.02	35.39	40.27	44.78	49.00	53.02	56.88	60.62	64.26	67.82
50	14.49	15.48	16.47	17.50	18.55	24.67	31.35	37.92	44.12	49.94	55.43	60.68	65.70	70.56	75.27	79.88
60	16.55	17.30	18.07	18.87	19.70	24.56	30.37	36.59	42.85	48.95	54.85	60.54	66.04	71.38	76.57	81.65
70	19.12	19.69	20.27	20.86	21.48	25.02	29.30	34.13	39.23	44.43	49.60	54.70	59.71	64.62	69.43	74.16
80	22.21	22.63	23.06	23.50	23.95	26.38	29.18	32.31	36.17	39.17	42.75	46.34	49.93	53.50	57.04	60.55
90	25.82	26.14	26.46	26.78	27.10	28.69	30.29	31.88	33.47	35.07	36.66	38.26	39.85	41.44	43.04	44.63
VALUES OF m AT ABUTMENT																
10	-0.245	-0.243	-0.236	-0.228	-0.220	-0.186	-0.164	-0.148	-0.137	-0.127	-0.120	-0.114	-0.109	-0.105	-0.101	-0.098
20	-0.769	-0.760	-0.764	-0.761	-0.752	-0.675	-0.609	-0.558	-0.520	-0.490	-0.465	-0.445	-0.429	-0.414	-0.402	-0.264
30	-1.745	-1.615	-1.515	-1.432	-1.363	-1.048	-0.817	-0.648	-0.521	-0.424	-0.348	-0.287	-0.237	-0.196	-0.161	-0.132
40	-3.183	-2.963	-2.756	-2.555	-2.357	-1.420	-0.649	-0.056	0.399	0.756	1.040	1.272	1.464	1.624	1.761	1.879
50	-5.080	-4.796	-4.513	-4.225	-3.942	-2.302	-0.684	0.783	1.993	2.994	3.823	4.526	5.120	5.631	6.074	6.462
60	-7.445	-7.111	-6.774	-6.425	-6.059	-3.922	-1.435	1.069	3.388	5.452	7.263	8.846	10.23	11.46	12.54	13.51
70	-10.29	-9.920	-9.543	-9.152	-8.741	-6.277	-3.170	0.282	3.774	7.114	10.21	13.05	15.62	17.95	20.06	21.99
80	-13.64	-13.24	-12.84	-12.42	-11.99	-9.330	-5.851	-1.723	2.737	7.291	11.73	15.97	19.96	23.68	27.13	30.34
90	-17.50	-17.09	-16.67	-16.24	-15.79	-13.06	-9.420	-4.912	0.217	5.695	11.29	16.84	22.23	27.41	32.34	37.00
VALUES OF v AT ABUTMENT																
10	-21.04	-21.80	-22.13	-22.40	-22.66	-23.94	-25.22	-26.52	-27.82	-29.13	-30.44	-31.76	-33.07	-34.39	-35.70	-37.02
20	-40.64	-41.18	-41.83	-42.42	-42.94	-45.15	-47.29	-49.48	-51.71	-53.99	-56.28	-58.60	-60.92	-63.26	-65.61	-67.96
30	-60.95	-61.02	-61.24	-61.53	-61.86	-63.69	-65.73	-68.01	-70.45	-73.00	-75.67	-78.39	-81.16	-83.97	-86.82	-89.68
40	-81.57	-81.67	-81.80	-81.92	-82.04	-82.50	-83.23	-84.42	-86.04	-87.96	-90.12	-92.46	-94.92	-97.50	-100.1	-102.9
50	-102.3	-102.6	-102.9	-103.1	-103.4	-103.4	-102.9	-102.4	-102.4	-102.8	-103.7	-104.8	-106.2	-107.8	-109.6	-111.4
60	-123.1	-123.6	-124.2	-124.7	-125.1	-126.1	-125.4	-124.0	-122.5	-121.3	-120.4	-120.0	-119.8	-119.9	-120.3	-120.8
70	-144.0	-144.8	-145.6	-146.4	-147.2	-149.6	-149.9	-148.8	-146.9	-144.8	-142.7	-140.8	-139.2	-137.8	-136.7	-135.8
80	-164.9	-166.1	-167.2	-168.3	-169.4	-173.5	-175.6	-175.8	-174.7	-172.8	-170.5	-168.0	-165.6	-163.3	-161.2	-159.3
90	-185.9	-187.4	-188.9	-190.3	-191.7	-197.7	-201.9	-204.1	-204.7	-204.1	-202.7	-200.9	-198.7	-196.4	-194.1	-191.9

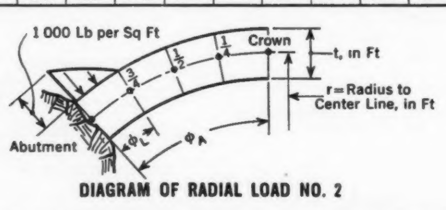


DIAGRAM OF RADIAL LOAD NO. 2

Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$

$M = mr^2$

$V = vr$

TABLE 4(*b*).—RADIAL DEFLECTIONS FOR RADIAL LOAD NO. 2

ϕ_A	VALUES OF t/r															
	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF K_1 AT CROWN															
10	-499.9	-114.1	-74.89	-61.34	-55.24	-48.35	-48.69	-50.24	-52.17	-54.28	-56.47	-58.72	-61.01	-63.32	-65.64	-67.98
20	227.8	-208.6	-181.2	-150.8	-131.4	-100.8	-97.09	-98.36	-101.2	-104.7	-108.6	-112.7	-116.9	-121.1	-125.5	-129.7
30	3720	331.7	-13.00	-89.20	-111.5	-122.9	-125.4	-129.6	-134.7	-140.2	-146.0	-151.9	-157.9	-163.9	-170.0	-176.1
40	12580	1719	504.8	183.7	60.25	-76.65	-108.3	-125.7	-137.9	-148.2	-157.4	-166.1	-174.4	-182.5	-190.4	-198.2
50	30800	4434	1474	682.3	355.0	25.97	-46.75	-79.82	-103.4	-121.3	-136.1	-148.1	-160.3	-170.9	-180.9	-190.5
60	63850	9204	3116	1494	864.5	176.0	55.03	-0.3403	-35.84	-61.98	-82.73	-100.0	-115.0	-128.4	-140.6	-151.9
70	118900	16990	5733	2752	1603	372.4	179.7	101.8	85.83	21.88	-3.959	-25.11	-43.04	-58.66	-72.56	-85.20
80	205300	29020	9713	4631	2621	628.4	329.4	221.2	161.7	122.6	92.92	69.19	49.48	32.64	17.96	4.873
90	334700	46870	15550	7353	41.48	963.6	510.0	358.5	283.1	236.2	202.9	177.6	157.3	140.3	125.9	113.4
	VALUES OF K_1 AT $\frac{1}{4}$ POINT															
10	-3971	-111.1	-736.3	-60.68	-54.90	-48.27	-48.46	-50.21	-52.22	-54.24	-56.53	-58.69	-61.06	-63.27	-65.70	-67.93
20	-63.74	-237.9	-187.2	-152.5	-131.9	-100.9	-97.16	-98.38	-101.2	-104.7	-108.5	-112.5	-116.7	-120.9	-125.2	-129.5
30	2097	96.48	-91.04	-125.7	-132.3	-127.5	-127.6	-131.0	-135.6	-140.8	-146.3	-151	-157.9	-163.8	-169.8	-175.8
40	7520	968.4	243.4	-54.70	-16.84	-98.08	-118.9	-132.0	-142.4	-151.5	-160.0	-168.1	-175.9	-183.6	-191.2	-198.7
50	18580	2645	856.0	378.0	177.1	-21.49	-71.40	-96.91	-116.1	-131.2	-144.1	-155.4	-165.8	-175.6	-184.9	-193.8
60	38480	5553	1873	890.2	508.4	85.62	5.504	-34.72	-62.22	-83.35	-100.6	-115.4	-128.4	-140.2	-151.0	-161.2
70	71380	10250	3470	1669	972.8	220.5	97.12	43.48	9.271	-16.26	-36.77	-53.96	-68.83	-81.99	-93.85	-104.8
80	122500	17440	5869	2814	1580	390.0	202.9	131.9	90.73	62.34	40.04	21.75	6.240	-7.256	-19.14	-29.89
90	198600	27990	9346	4449	2575	605.5	326.0	229.8	180.2	148.2	125.9	106.4	91.28	78.55	67.62	57.99
	VALUES OF K_1 AT $\frac{1}{2}$ POINT															
10	-307.8	-101.6	-69.69	-58.65	-53.64	-48.03	-48.72	-50.11	-52.20	-54.14	-56.46	-58.57	-60.96	-63.14	-65.56	-67.78
20	-707.7	-296.9	-197.0	-154.2	-132.1	-100.9	-97.26	-98.39	-101.1	-104.4	-108.2	-112.1	-115.6	-120.3	-124.5	-128.7
30	-1569	-437.4	-269.1	-209.7	-179.8	-138.9	-133.4	-134.6	-138.0	-142.3	-147.3	-152.5	-157.9	-163.4	-169.0	-174.7
40	-3879	-741.6	-358.9	-246.7	-199.6	-147.1	-144.9	-149.0	-154.6	-160.6	-167.0	-173.4	-180.0	-186.6	-193.2	-199.9
50	-8833	-1406	-559.9	-327.9	-244.5	-140.1	-136.9	-142.3	-150.2	-158.2	-166.0	-173.6	-181.0	-188.4	-195.4	-202.8
60	-18080	-2652	-948.0	-494.5	-319.0	-136.7	-121.2	-124.8	-132.4	-140.8	-149.1	-157.1	-164.9	-172.4	-179.7	-186.8
70	-33880	-4778	-1616	-788.8	-473.0	-147.0	-100.3	-106.8	-111.0	-117.4	-113.1	-131.6	-138.5	-145.2	-151.7	-158.0
80	-59250	-8181	-2689	-1265	-635.8	-176.7	-109.5	-94.62	-92.91	-95.25	-99.43	-104.2	-109.1	-114.0	-118.8	-123.5
90	-98200	-13390	-4330	-1997	-1121	-231.8	-120.8	-90.98	-81.23	-78.49	-78.68	-80.16	-82.23	-84.54	-86.92	-89.33
	VALUES OF K_1 AT $\frac{3}{4}$ POINT															
10	-206.9	-84.00	-62.65	-55.06	-51.53	-47.61	-48.33	-49.95	-51.87	-53.98	-56.14	-58.38	-60.64	-62.92	-65.22	-67.54
20	-1015	-299.6	-185.8	-145.5	-126.3	-100.2	-97.13	-98.26	-100.8	-104.0	-107.6	-111.4	-115.3	-119.3	-123.4	-127.5
30	-3719	-760.6	-380.9	-265.3	-214.9	-150.2	-140.2	-139.1	-141.0	-144.3	-148.4	-152.8	-157.6	-162.6	-167.7	-172.9
40	-10550	-1815	-767.8	-468.7	-345.1	-198.6	-176.0	-170.7	-170.7	-173.0	-176.5	-180.7	-185.4	-190.4	-195.7	-201.1
50	-24540	-3888	-1496	-833.0	-569.4	-260.6	-213.3	-198.9	-194.5	-194.0	-195.7	-198.4	-201.9	-206.0	-210.4	-215.0
60	-49750	-7525	-2740	-1440	-925.3	-351.0	-262.0	-232.9	-220.7	-215.2	-213.2	-213.1	-214.3	-216.1	-218.7	-221.6
70	-91360	-13420	-4718	-2386	-1476	-480.5	-332.6	-279.7	-256.5	-244.0	-254.5	-232.6	-261.8	-229.2	-269.0	-229.4
80	-155900	-22430	-7703	-3795	-2288	-659.7	-422.3	-343.2	-306.2	-385.1	-271.8	-262.8	-256.4	-251.7	-248.3	-245.9
90	-251500	-35640	-12030	-5815	-3434	-901.4	-542.8	-425.8	-375.2	-340.6	-320.7	-306.7	-296.2	-288.1	-281.6	-276.3
	VALUES OF K_1 AT ABUTMENT															
10	-37.55	-38.92	-39.50	-39.99	-40.46	-42.73	-45.02	-47.34	-49.67	-52.00	-54.34	-56.69	-59.03	-61.38	-63.73	-66.08
20	-72.53	-73.50	-74.66	-75.71	-76.64	-80.59	-84.41	-88.32	-92.31	-96.36	-100.5	-104.6	-108.8	-112.9	-117.1	-121.3
30	-108.8	-108.9	-109.3	-109.8	-110.4	-113.7	-117.3	-121.4	-125.7	-130.3	-135.1	-139.9	-144.9	-149.9	-155.0	-160.1
40	-145.6	-145.8	-146.0	-146.2	-146.4	-147.3	-148.6	-150.7	-153.6	-157.0	-160.9	-165.0	-169.4	-174.0	-178.8	-183.6
50	-182.6	-183.1	-183.6	-184.1	-184.5	-184.6	-183.6	-182.8	-182.8	-183.6	-185.1	-187.1	-189.6	-192.4	-195.6	-198.9
60	-219.7	-220.7	-221.7	-222.6	-223.3	-225.0	-223.8	-221.3	-218.7	-216.5	-215.0	-214.1	-213.8	-214.0	-214.7	-215.7
70	-256.9	-258.5	-260.0	-261.4	-262.7	-267.0	-267.6	-265.6	-262.2	-258.4	-254.7	-251.3	-248.4	-246.0	-244.0	-242.5
80	-294.3	-296.4	-298.5	-300.4	-302.3	-309.7	-313.5	-313.9	-311.9	-308.4	-304.3	-299.9	-295.6	-291.6	-287.8	-284.3
90	-331.9	-334.5	-337.1	-339.7	-342.2	-352.9	-360.4	-364.3	-365.4	-364.4	-361.9	-358.5	-354.7	-350.6	-346.5	-342.5

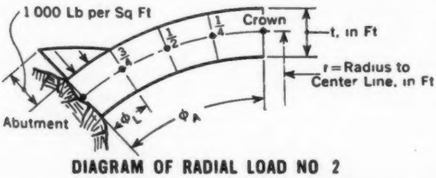


DIAGRAM OF RADIAL LOAD NO 2

Radial Deflection, in Ft

$$\Delta_t = K_1 \frac{r}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 5(*a*).—FORCES AND MOMENTS FOR RADIAL LOAD NO. 3

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF h AT CROWN															
10	22.85	11.32	8.388	7.387	7.014	7.146	7.741	8.330	8.893	9.429	9.951	10.46	10.96	11.46	11.95	12.43
20	57.27	48.56	40.40	34.91	31.51	27.19	28.30	30.26	32.37	34.49	36.57	38.62	40.63	42.62	44.58	46.58
30	61.73	62.89	62.07	60.41	58.64	54.73	56.69	60.55	64.96	69.50	74.04	78.53	82.97	87.34	89.84	95.95
40	62.30	65.52	67.91	69.69	71.07	76.56	83.16	90.82	98.82	106.9	114.8	122.6	130.3	137.9	145.4	152.8
50	62.69	66.14	69.28	72.17	74.89	87.56	100.3	113.3	126.2	138.8	151.1	163.2	174.9	186.5	197.8	208.9
60	63.31	66.64	69.84	72.96	76.05	91.69	108.4	125.9	143.6	161.0	178.1	194.7	211.0	226.9	242.5	257.9
70	64.21	67.36	70.46	73.56	76.66	93.02	111.3	131.3	152.2	173.5	194.7	215.6	236.3	256.5	276.4	296.0
80	65.38	68.37	71.35	74.35	77.38	93.54	112.0	132.8	155.3	178.9	202.9	227.2	251.3	275.4	299.1	322.6
90	66.81	69.68	72.55	75.45	78.38	94.09	112.2	132.8	155.6	180.0	205.6	231.9	258.6	285.3	312.1	338.9
	VALUES OF m AT CROWN															
10	.0958	0.216	0.286	0.342	0.391	0.586	0.739	0.870	0.986	1.093	1.193	1.289	1.380	1.469	1.555	1.639
20	-0.450	-0.182	.0905	0.313	0.491	1.071	1.472	1.815	2.127	2.420	2.699	2.967	3.228	3.481	3.729	4.215
30	-1.249	-1.195	-1.044	-0.846	-0.637	0.201	0.715	1.084	1.390	1.661	1.913	2.153	2.384	2.608	2.828	3.043
40	-2.256	2.369	-2.421	-2.426	-2.401	-2.170	-2.090	-2.174	-2.350	-2.575	-2.828	-3.096	-3.373	-3.655	-3.941	-4.229
50	-3.512	-3.743	-3.946	-4.129	-4.300	-5.146	-6.178	-7.402	-8.777	-10.16	-11.61	-13.09	-14.57	-16.06	-17.55	-19.05
60	-5.030	-5.360	-5.680	-6.000	-6.326	-8.180	-10.54	-13.35	-16.44	-19.71	-23.08	-26.51	-29.98	-33.47	-36.98	-40.50
70	-6.820	-7.239	-7.661	-8.095	-8.546	-11.22	-14.77	-19.15	-24.18	-29.64	-35.40	-41.36	-47.45	-53.64	-59.90	-66.21
80	-8.884	-9.390	-9.908	-10.44	-11.00	-14.34	-18.84	-24.56	-31.33	-38.94	-47.18	-55.87	-64.91	-74.19	-83.68	-93.31
90	-11.22	-11.82	-12.43	-13.06	-13.72	-17.62	-22.87	-29.62	-37.81	-47.25	-57.71	-68.99	-80.91	-93.34	-106.2	-119.3
	VALUES OF h AT ABUTMENT															
10	23.78	12.45	9.577	8.607	8.256	8.465	9.130	9.790	10.42	11.03	11.62	12.20	2.78	13.34	13.91	14.46
20	58.95	50.83	43.23	38.13	35.00	31.26	32.62	34.77	37.07	39.38	41.65	43.89	46.10	48.28	50.45	52.65
30	64.99	66.13	65.56	64.27	62.88	60.21	62.61	66.67	71.20	75.85	80.49	85.09	89.64	94.14	97.02	103.0
40	68.16	70.88	72.96	74.58	75.89	81.36	87.67	94.80	102.2	109.6	117.0	124.2	131.4	138.5	145.4	152.4
50	72.12	74.73	77.14	79.39	81.54	91.64	101.8	112.1	122.4	132.4	142.3	152.0	161.6	170.9	180.2	189.3
60	77.29	79.52	81.68	83.81	85.91	96.55	107.7	119.3	130.9	142.5	153.8	165.0	175.9	186.7	197.3	207.8
70	83.77	85.61	87.43	89.25	91.08	100.5	110.6	121.2	132.2	143.3	154.6	165.3	176.2	186.9	197.6	208.1
80	91.62	93.13	94.64	96.15	97.67	105.4	113.6	122.2	131.0	140.1	149.2	158.4	167.5	176.6	185.7	94.8
90	100.9	102.2	103.4	104.7	105.9	112.1	118.4	124.6	130.8	137.1	143.3	149.5	155.8	162.0	168.2	174.4
	VALUES OF m AT ABUTMENT															
10	-0.842	-0.913	-0.903	-0.878	-0.851	-0.733	-0.650	-0.589	-0.544	-0.508	-0.479	-0.456	-0.436	-0.419	-0.405	-0.392
20	-2.128	-2.449	-2.732	-2.904	-2.995	-2.992	-2.841	-2.696	-2.574	-2.470	-2.383	-2.308	-2.243	-2.186	-2.137	-1.846
30	-4.505	-4.433	-4.540	-4.707	-4.875	-5.274	-5.209	-5.033	-4.849	-4.680	-4.531	-4.402	-4.288	-4.189	-4.099	-4.024
40	-8.117	-7.729	-7.475	-7.317	-7.219	-6.966	-6.604	-6.158	-5.722	-5.327	-4.981	-4.679	-4.416	-4.187	-3.985	-3.806
50	-12.94	-12.33	-11.81	-11.35	-10.94	-9.228	-7.677	-6.230	-4.960	-3.792	-2.809	-1.940	-1.187	-0.526	0.058	0.577
60	-19.01	-18.24	-17.52	-16.84	-16.19	-13.04	-9.855	-6.733	-3.820	-1.186	1.164	3.250	5.103	6.755	8.230	9.562
70	-26.38	-25.49	-24.63	-23.79	-22.96	-18.69	-14.00	-9.048	-4.123	0.575	5.201	8.955	12.61	15.95	18.99	21.76
80	-35.12	-34.15	-33.19	-32.24	-31.29	-26.22	-20.39	-13.88	-7.021	-0.136	6.558	12.93	18.93	24.53	29.74	34.58
90	-45.34	-44.32	-43.30	-42.28	-41.25	-35.68	-29.09	-21.46	-13.07	-4.271	4.622	13.39	21.89	30.02	37.77	45.10
	VALUES OF v AT ABUTMENT															
10	-40.18	-42.73	-43.78	-44.50	-45.11	-47.82	-50.44	-53.06	-55.69	-58.32	-60.95	-63.59	-66.23	-68.87	-71.51	-74.15
20	-68.55	-72.61	-76.49	-79.46	-81.71	-88.62	-93.69	-98.46	-103.2	-107.9	-112.6	-117.4	-122.1	-126.9	-131.6	-136.4
30	-100.9	-102.0	-104.0	-106.5	-109.0	-119.1	-126.2	-132.4	-138.3	-144.2	-150.1	-156.0	-161.9	-167.8	-174.7	-179.8
40	-134.9	-135.0	-135.6	-136.6	-137.9	-145.2	-151.7	-157.6	-163.2	-168.9	-174.6	-180.3	-186.2	-192.1	-198.1	-204.1
50	-169.4	-169.4	-169.7	-170.2	-170.8	-174.5	-178.2	-181.6	-185.2	-188.9	-192.9	-197.1	-201.5	-206.1	-210.8	-215.7
60	-204.2	-204.6	-205.0	-205.5	-206.0	-208.4	-210.0	-210.8	-211.5	-212.4	-213.6	-215.2	-217.1	-219.3	-221.7	-224.4
70	-239.4	-240.2	-240.9	-241.7	-242.5	-245.6	-246.9	-246.7	-245.5	-244.0	-242.6	-241.4	-240.6	-240.0	-239.8	-239.9
80	-274.9	-276.2	-277.4	-278.6	-279.9	-284.9	-287.5	-288.1	-286.9	-284.6	-281.9	-279.0	-276.1	-273.4	-270.9	-268.7
90	-310.8	-312.6	-314.4	-316.1	-317.8	-325.4	-330.7	-333.4	-333.9	-332.7	-330.4	-327.5	-324.1	-320.6	-317.2	-313.7

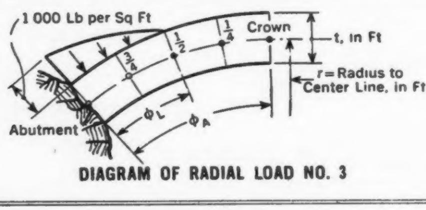


DIAGRAM OF RADIAL LOAD NO. 3

Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$
$M = mr^2$
$V = vr$

TABLE 5(*b*).—Radial Deflections for Radial Load No. 3

ϕ_A	Values of t/r: .025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
Values of K_1 at Crown																
10	-1614	-456.5	-246.3	-177.3	-146.6	-109.0	-104.9	-106.1	-108.9	-112.5	-116.5	-120.6	-125.0	-129.4	-133.9	-138.5
20	-374.3	-1337	-904.9	-642.8	-494.8	-272.7	-233.4	-224.1	-220.8	-227.6	-233.5	-239.8	-247.1	-254.8	-262.8	-270.8
30	14770	489.8	-600.1	-701.0	-654.4	-423.9	-351.8	-329.2	-323.7	-325.7	-331.5	-339.4	-348.6	-358.6	-369.0	-380.3
40	55080	6197	1271	132.9	-228.4	-419.9	-390.0	-376.4	-374.9	-379.7	-387.9	-398.3	-409.9	-422.4	-435.5	-449.0
50	139900	17970	5168	1974	824.4	-211.5	-305.3	-332.9	-351.8	-367.3	-383.4	-399.4	-415.5	-431.8	-448.0	-464.3
60	295900	39250	12080	5194	2652	186.2	-104.8	-198.2	-248.3	-283.5	-312.1	-337.0	-359.8	-381.1	-401.4	-421.0
70	558800	74640	23430	10400	5564	784.7	197.6	13.14	-78.28	-136.9	-180.6	-216.0	-237.5	-273.3	-298.0	-320.9
80	974700	130200	41070	18400	9984	1634	606.0	293.4	148.1	61.25	0.733	-45.74	-83.75	-116.2	-144.7	-170.4
90	1603000	213700	67390	29680	16470	2814	1143	647.4	428.3	305.2	224.5	165.9	120.2	88.77	51.10	23.47
Values of K_1 at $\frac{1}{4}$ Point																
10	-1543	-437.1	-238.7	-173.5	-144.5	-108.6	-104.7	-105.9	-108.8	-112.4	-116.4	-120.6	-124.7	-129.3	-133.8	-138.4
20	-1615	-1402	-897.1	-630.4	-484.8	-270.1	-232.2	-223.5	-223.5	-227.2	-232.9	-239.3	-246.6	-254.3	-262.2	-270.3
30	6935	-465.0	-855.9	-793.7	-693.9	-425.9	-352.4	-329.6	-323.9	-325.7	-331.3	-338.9	-348.0	-357.8	-368.3	-379.1
40	29620	2811	225.1	-321.5	-469.4	-455.1	-407.0	-384.6	-383.5	-383.5	-394.1	-400.1	-414.7	-423.0	-439.6	-448.6
50	76930	9496	2483	765.7	165.4	-326.9	-354.5	-362.3	-371.9	-382.6	-395.4	-408.9	-423.2	-437.8	-452.8	-468.0
60	163300	21450	6428	2636	1247	-70.18	-215.4	-266.9	-297.1	-321.2	-342.5	-362.3	-381.2	-399.4	-417.3	-434.7
70	307800	41110	12810	5605	2934	307.1	-9.847	-116.3	-171.7	-210.3	-241.2	-267.6	-291.3	-313.1	-333.5	-352.9
80	534600	71680	22620	10100	5449	826.1	255.2	77.92	-8.391	-63.07	-103.4	-136.0	-163.7	-188.2	-210.4	-230.8
90	874500	117200	37090	16670	9083	1528	328.0	314.4	186.9	112.6	52.71	23.25	-7.828	-34.08	-56.91	-77.33
Values of K_1 at $\frac{1}{2}$ Point																
10	-1299	-376.8	-215.5	-161.9	-137.7	-107.3	-104.2	-105.6	-108.5	-112.1	-116.1	-120.3	-124.6	-129.0	-133.5	-138.1
20	-4056	-1461	-840.4	-581.0	-449.3	-261.8	-229.1	-221.6	-222.0	-225.9	-231.4	-237.9	-245.1	-252.7	-263.5	-268.5
30	-9509	-2430	-1360	-963.9	-758.8	-425.6	-352.3	-329.7	-323.8	-325.2	-330.3	-337.4	-345.9	-355.2	-365.2	-375.6
40	-23970	-4344	-1989	-1284	-972.2	-532.1	-436.5	-404.3	-394.0	-393.3	-397.6	-404.9	-413.9	-424.1	-435.2	-446.8
50	-55040	-8399	-3234	-1828	-1263	-590.8	-534.9	-435.7	-423.9	-422.4	-426.7	-434.0	-443.3	-453.9	-465.3	-477.4
60	-113100	-15960	-5561	-2838	-1791	-655.9	-487.8	-438.0	-421.9	-419.0	-422.4	-429.3	-438.2	-448.4	-459.4	-471.1
70	-212200	-28880	-9531	-4569	-2702	-771.7	-510.6	-434.5	-407.4	-398.8	-398.8	-403.1	-410.0	-418.4	-427.8	-437.8
80	-371000	-49530	-15880	-7342	-3145	-972.4	-562.7	-443.0	-396.8	-377.5	-370.5	-369.7	-372.4	-377.1	-383.2	-390.2
90	-613700	-81020	-25560	-11580	-6411	-1293	-660.1	-476.1	-401.7	-366.6	-349.0	-340.4	-336.9	-336.3	-337.6	-340.3
Values of K_1 at $\frac{3}{4}$ Point																
10	-779.9	-261.6	-169.4	-137.3	-122.3	-103.0	-101.8	-103.8	-107.0	-110.8	-114.9	-119.1	-123.5	-127.9	-132.4	-137.0
20	-3978	-1094	-616.3	-441.7	-357.0	-237.7	-217.1	-213.4	-215.5	-220.1	-226.0	-232.7	-239.9	-247.5	-255.2	-263.2
30	-14380	-2770	-1310	-862.0	-661.7	-386.2	-332.3	-316.4	-313.1	-315.5	-320.8	-327.9	-336.1	-345.0	-354.6	-364.4
40	-41310	-6668	-2668	-1549	-996.0	-535.6	-436.4	-403.7	-392.3	-390.1	-392.7	-398.3	-405.6	-413.4	-423.6	-433.7
50	-97470	-14550	-5303	-2808	-1825	-718.8	-541.3	-482.3	-458.0	-448.0	-445.6	-447.2	-451.7	-458.0	-465.6	-474.1
60	-199900	-28670	-9932	-4975	-3058	-984.8	-674.1	-571.2	-524.0	-503.1	-491.9	-487.1	-486.3	-488.2	-492.2	-497.2
70	-370800	-51940	-17460	-8449	-5006	-1378	-859.1	-689.8	-612.7	-571.6	-546.5	-533.4	-524.9	-520.3	-518.4	-518.5
80	-638300	-88030	-29020	-13730	-7946	-1942	-1117	-852.6	-732.3	-666.3	-625.9	-599.3	-580.3	-568.4	-559.4	-553.2
90	-1038000	-141600	-46050	-21460	-12210	-2730	-1467	-1071	-893.1	-795.5	-734.9	-693.9	-664.7	-642.8	-626.1	-613.0
Values of K_1 at Abutment																
10	-71.73	-76.27	-78.15	-79.44	-80.53	-85.35	-90.03	-94.71	-99.40	-104.1	-108.8	-113.5	-118.2	-122.9	-127.6	-132.4
20	-122.4	-129.6	-136.5	-141.8	-145.8	-158.2	-167.2	-175.7	-184.2	-192.6	-201.0	-209.5	-218.0	-226.5	-235.0	-243.5
30	-180.1	-182.0	-185.6	-190.0	-194.5	-212.5	-225.3	-236.4	-247.0	-257.4	-267.9	-278.4	-289.0	-299.6	-310.2	-320.9
40	-240.8	-240.9	-242.0	-243.9	-264.0	-259.1	-270.8	-281.3	-291.4	-301.4	-311.6	-321.9	-332.3	-342.9	-353.6	-364.4
50	-302.4	-302.4	-302.9	-303.8	-304.8	-311.5	-318.0	-324.8	-330.5	-337.2	-344.4	-351.8	-359.7	-367.9	-376.4	-385.1
60	-364.6	-365.1	-365.9	-366.8	-367.7	-372.1	-374.8	-376.3	-377.5	-379.1	-381.3	-384.1	-387.4	-391.4	-395.8	-400.6
70	-427.4	-428.7	-430.1	-431.5	-432.9	-438.5	-440.8	-440.3	-438.2	-435.6	-433.1	-431.0	-429.4	-428.5	-428.1	-428.2
80	-490.7	-492.9	-495.2	-497.4	-499.6	-508.5	-513.4	-514.2	-512.1	-508.1	-503.2	-498.0	-492.8	-488.0	-483.6	-479.6
90	-544.7	-557.9	-561.1	-564.3	-567.4	-580.9	-590.3	-595.1	-596.0	-593.9	-589.8	-584.5	-578.6	-572.4	-566.1	-560.0

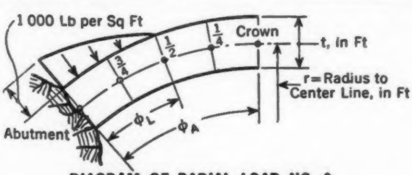


DIAGRAM OF RADIAL LOAD NO. 3

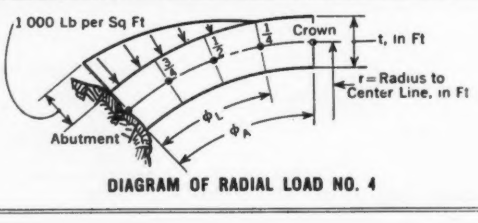
Radial Deflection, in Ft

$$\Delta_r = K_1 \frac{r}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 6(*a*).—FORCES AND MOMENTS FOR RADIAL LOAD NO. 4

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF h AT CROWN																
10	56.39	24.62	16.38	13.33	11.97	10.83	11.31	11.97	12.66	13.34	14.01	14.68	15:35	16.01	16.67	17.32
20	150.7	119.8	93.43	75.86	64.69	46.42	44.63	45.89	48.01	50.42	52.95	55.52	58.10	60.69	63.27	65.98
30	165.9	161.8	153.0	142.6	132.7	103.8	96.81	97.45	100.9	105.5	110.6	115.9	121.4	127.0	132.5	138.1
40	169.0	172.0	172.6	171.5	169.4	157.0	152.7	155.2	161.1	168.8	177.3	186.3	195.6	205.0	214.4	223.9
50	170.9	175.5	179.1	181.9	184.0	189.8	195.7	204.6	215.7	228.3	241.6	255.5	269.6	283.7	297.9	312.1
60	173.0	178.0	182.5	186.6	190.4	206.7	222.2	238.8	256.8	275.6	296.0	314.6	334.3	354.0	373.6	393.2
70	175.6	180.6	185.4	189.9	194.4	215.6	237.0	259.7	283.7	308.6	333.9	359.5	385.1	410.6	436.1	461.4
80	178.7	183.6	188.4	193.2	198.2	221.2	245.7	272.1	300.2	329.6	359.9	390.6	421.6	452.6	483.5	514.3
90	182.4	187.2	192.0	196.8	201.6	225.8	251.7	280.0	310.4	342.8	376.4	411.0	446.2	481.7	517.4	553.1
VALUES OF m AT CROWN																
10	0.357	0.654	0.817	0.943	1.052	1.490	1.834	2.129	2.394	2.637	2.866	3.084	3.295	3.499	3.698	3.893
20	-0.781	.0394	0.818	1.433	1.916	3.452	4.508	5.411	6.236	7.011	7.750	8.463	9.156	9.832	10.49	11.49
30	-2.499	-2.112	-1.469	-0.714	0.052	3.080	5.047	6.556	7.856	9.044	10.17	11.24	12.29	13.31	14.31	15.30
40	-4.642	-4.611	-4.383	-4.001	-3.513	-0.679	1.641	3.362	4.716	5.855	6.865	7.793	8.666	9.502	10.31	11.10
50	-7.313	-7.521	-7.608	-7.590	-7.484	-6.274	-4.967	-4.059	-3.532	-3.279	-3.200	-3.273	-3.420	-3.627	-3.877	-4.159
60	-10.56	-10.95	-11.27	-11.53	-11.74	-12.37	-12.94	-13.87	-15.19	-16.85	-18.75	-20.83	-23.04	-25.34	-27.72	-30.16
70	-14.40	-14.95	-15.47	-15.95	-16.42	-18.66	-21.22	-24.40	-28.20	-32.52	-37.25	-42.28	-47.54	-52.97	-58.54	-64.21
80	-18.85	-19.56	-20.25	-20.93	-21.70	-25.20	-29.54	-34.90	-41.27	-48.53	-56.50	-65.02	-73.98	-83.29	-92.86	-102.6
90	-23.92	-24.79	-25.65	-26.51	-27.39	-32.14	-37.99	-45.24	-53.95	-63.98	-75.15	-87.26	-100.1	-113.6	-127.6	-142.0
VALUES OF h AT ABUTMENT																
10	58.42	27.17	19.09	16.12	14.82	13.88	14.53	15.36	16.21	17.06	17.90	18.74	19.57	20.40	21.23	22.05
20	153.2	124.2	99.60	83.24	72.88	56.43	55.45	57.35	60.05	63.03	66.12	69.25	72.39	75.53	78.66	81.92
30	169.5	166.3	158.9	150.2	142.0	118.6	114.1	116.3	120.8	126.4	132.4	138.7	145.0	151.4	157.8	164.2
40	175.1	177.9	179.0	178.7	177.6	171.0	170.5	175.2	182.6	191.2	200.6	210.3	220.2	230.2	240.3	250.4
50	180.6	184.5	187.6	190.3	192.5	200.6	208.8	218.9	230.4	242.8	255.8	269.1	282.5	295.9	309.4	322.9
60	187.4	191.2	194.7	198.0	201.1	225.5	229.5	244.0	259.2	274.9	291.3	306.8	322.9	339.0	355.0	371.1
70	195.9	199.3	202.6	205.8	209.0	224.7	240.4	256.6	273.1	290.0	307.5	324.2	341.3	358.5	375.6	392.6
80	206.2	209.2	212.2	215.2	218.2	233.0	248.1	263.5	279.2	295.1	311.2	327.3	343.5	359.7	375.9	392.1
90	218.5	221.2	223.9	226.6	229.3	242.8	256.2	269.7	283.2	296.7	310.2	323.7	337.2	350.6	364.1	377.6
VALUES OF m AT ABUTMENT																
10	-1.675	-1.897	-1.895	-1.851	-1.797	-1.556	-1.382	-1.256	-1.169	-1.084	-1.023	-0.972	-0.950	-0.895	-0.864	-0.837
20	-3.218	-4.405	-5.358	-5.946	-6.279	-6.555	-6.319	-6.052	-5.811	-5.602	-5.421	-5.265	-5.128	-5.008	-4.901	-4.452
30	-6.098	-6.578	-7.438	-8.392	-9.275	-11.71	-12.27	-12.27	-12.10	-11.90	-11.68	-11.48	-11.30	-11.13	-10.98	-10.84
40	-10.73	-10.57	-10.77	-11.21	-11.78	-14.65	-16.15	-16.68	-16.75	-16.64	-16.44	-16.22	-16.00	-15.79	-15.58	-15.40
50	-17.02	-16.45	-16.12	-16.00	-16.02	-17.09	-18.03	-18.33	-18.18	-17.80	-17.34	-16.82	-16.32	-15.84	-15.38	-14.96
60	-24.99	-24.13	-23.44	-22.89	-22.45	-21.18	-20.24	-19.05	-17.63	-16.09	-14.05	-13.05	-11.64	-10.33	-9.111	-7.976
70	-34.73	-33.67	-32.72	-31.86	-31.08	-27.77	-24.61	-21.21	-17.62	-13.98	-9.978	-7.022	-3.821	-0.830	1.954	4.541
80	-46.37	-45.16	-44.03	-42.96	-41.78	-37.06	-31.95	-26.31	-20.25	-14.01	-7.785	-1.740	4.055	9.552	14.74	19.61
90	-60.06	-58.76	-57.50	-56.29	-55.10	-49.13	-42.53	-35.03	-26.74	-17.94	-8.920	.0813	8.903	17.44	25.63	33.44
VALUES OF v AT ABUTMENT																
10	-56.38	-62.72	-64.96	-66.31	-67.36	-71.64	-75.65	-79.62	-83.58	-87.55	-91.52	-95.48	-99.45	-103.4	-107.4	-111.4
20	-80.23	-92.44	-103.1	-110.7	-116.2	-130.6	-139.3	-147.0	-154.4	-161.7	-169.0	-176.2	-183.5	-190.7	-198.0	-205.2
30	-113.3	-117.8	-124.6	-132.2	-139.6	-166.2	-181.8	-193.6	-204.0	-213.8	-223.4	-232.8	-242.2	-251.5	-260.8	-270.2
40	-150.4	-151.7	-154.6	-158.4	-163.0	-186.9	-205.7	-220.1	-232.3	-243.4	-253.9	-264.0	-274.1	-284.0	-294.0	-303.9
50	-188.8	-189.2	-190.3	-192.2	-194.5	-209.8	-225.0	-237.9	-249.1	-259.2	-268.8	-277.9	-286.8	-295.7	-304.6	-313.4
60	-227.8	-228.1	-228.8	-230.0	-231.3	-240.5	-250.4	-259.3	-267.1	-274.0	-279.7	-286.9	-293.2	-299.4	-305.7	-312.0
70	-267.3	-268.0	-268.8	-269.8	-271.0	-277.8	-284.3	-289.6	-293.8	-297.1	-300.0	-302.7	-305.3	-308.0	-310.8	-313.7
80	-307.5	-308.6	-309.8	-311.1	-312.2	-319.3	-325.0	-328.9	-331.0	-331.9	-331.9	-331.5	-330.9	-330.2	-329.6	-329.0
90	-348.2	-349.9	-351.6	-353.4	-355.2	-363.7	-370.5	-375.0	-377.3	-377.7	-376.8	-375.0	-372.5	-369.8	-366.9	-363.9



Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$

$M = mr^2$

$V = vr$

DIAGRAM OF RADIAL LOAD NO. 4

TABLE 6(*b*).—Radial Deflections for Radial Load No. 4

VALUES OF K₁ AT CROWN

ϕ_A	VALUES OF t/r															
	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-4316	-1078	-527.7	-352.7	-276.4	-182.1	-168.6	-167.4	-170.2	-174.6	-179.9	-185.7	-191.9	-198.3	-204.8	-211.5
20	-4885	-3974	-2383	-1572	-1141	-521.2	-410.2	-377.6	-370.3	-368.5	-373.5	-381.1	-390.4	-400.7	-411.8	-423.0
30	21420	-1507	-2514	-2216	-1850	-937.7	-690.0	-603.1	-569.1	-557.5	-557.0	-562.6	-572.0	-583.8	-597.3	-612.0
40	93330	-8255	441.3	-1083	-1418	-1131	-881.4	-770.5	-720.9	-700.5	-695.3	-698.9	-707.9	-720.6	-735.6	-752.4
50	245700	28910	7088	1937	221.0	-944.9	-875.5	-807.1	-771.1	-756.5	-754.9	-761.1	-772.6	-787.6	-805.0	-824.2
60	527400	66620	19120	7422	3263	-408.4	-657.9	-690.9	-698.3	-706.4	-712.3	-734.0	-752.2	-772.4	-794.0	-816.7
70	1003000	129800	39050	16420	-8212	522.9	-249.1	-436.0	-510.6	-554.6	-588.5	-618.5	-647.0	-674.7	-711.9	-729.1
80	1759000	229300	70240	30390	16020	1882	351.8	-55.27	-222.4	-314.2	-376.0	-424.7	-465.7	-502.2	-535.9	-567.8
90	2902000	379400	117000	51200	27090	3815	1176	456.2	161.6	6.154	-91.88	-162.5	-217.8	-264.1	-304.4	-340.8

VALUES OF K₁ AT ¼ POINT

ϕ_A	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-4053	-1018	-505.9	-342.0	-270.0	-181.0	-168.2	-167.2	-170.0	-174.4	-179.8	-185.6	-191.7	-198.1	-204.6	-211.3
20	-6963	-3954	-2298	-1510	-1098	-511.8	-406.5	-375.6	-366.9	-367.4	-372.5	-380.2	-389.4	-399.7	-410.7	-422.0
30	6153	-3150	-2866	-2298	-1857	-922.7	-686.1	-599.1	-566.4	-555.3	-554.9	-560.6	-569.9	-581.6	-594.9	-609.4
40	42350	1833	-1409	-1820	-1766	-1155	-885.7	-772.1	-721.8	-700.7	-695.0	-698.1	-706.6	-718.6	-733.1	-749.3
50	118400	12370	2043	-234.0	-906.9	-1095	-926.6	-834.0	-788.6	-768.9	-764.1	-767.8	-777.4	-790.7	-806.6	-824.6
60	258100	31380	8222	2637	713.5	-797.4	-806.4	-773.0	-753.2	-746.9	-744.4	-759.4	-772.9	-789.3	-811.4	-827.7
70	492600	62910	18330	7275	3308	-263.1	-557.7	-610.7	-629.4	-644.1	-660.0	-677.9	-697.2	-717.9	-747.9	-761.8
80	862100	112200	33960	14370	6678	494.5	-196.9	-367.9	-437.0	-447.7	-509.0	-536.6	-562.5	-587.5	-612.0	-636.3
90	1418000	185800	57170	24810	12950	1544	283.2	-51.13	-186.9	-260.6	-314.0	-348.5	-381.0	-410.0	-436.8	-462.2

VALUES OF K₁ AT ½ POINT

ϕ_A	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-3180	-831.0	-435.4	-306.7	-248.9	-176.3	-165.8	-165.6	-168.7	-173.3	-178.7	-184.6	-190.8	-197.2	-203.8	-210.4
20	-9897	-3548	-1954	-1287	-950.7	-477.3	-391.2	-366.2	-360.0	-361.6	-367.3	-375.3	-384.7	-395.0	-406.0	-417.3
30	-21290	-5799	-3265	-2272	-1742	-853.8	-649.2	-578.6	-551.6	-543.0	-543.9	-550.0	-559.5	-571.1	-584.2	-598.3
40	-51040	-9779	-4647	-3039	-2293	-1150	-866.3	-757.2	-709.7	-689.8	-684.1	-686.7	-694.4	-705.4	-718.8	-733.9
50	-114900	-18070	-7212	-4190	-2939	-1340	-1002	-870.6	-810.5	-782.7	-772.2	-771.4	-777.0	-786.8	-799.5	-814.3
60	-234200	-33500	-1194	-6250	-4031	-1528	-1087	-929.6	-858.1	-824.0	-804.6	-805.5	-808.3	-816.8	-828.0	-841.4
70	-437400	-59760	-19970	-9744	-5873	-1782	-1173	-968.1	-877.4	-833.0	-811.6	-803.3	-802.8	-807.7	-822.1	-827.0
80	-762000	-101600	-32780	-15320	-8654	-2198	-1304	-1019	-895.3	-833.2	-800.7	-784.3	-777.6	-777.1	-780.8	-787.4
90	-1257000	-165400	-52270	-23800	-13290	-2841	-1486	-1107	-933.3	-844.3	-794.8	-766.4	-750.3	-742.0	-738.9	-739.5

VALUES OF K₁ AT ¾ POINT

ϕ_A	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-1627	-449.3	-300.7	-232.9	-201.7	-161.0	-156.7	-158.8	-163.1	-168.4	-174.3	-180.5	-187.0	-193.5	-200.2	207.0
20	-7014	-2093	-1172	-816.8	-684.1	-390.2	-344.7	-333.9	-334.3	-339.7	-347.6	-357.0	-367.4	-387.4	-389.9	-401.6
30	-22640	-4661	-2303	-1540	-1181	-653.3	-541.3	-505.1	-494.2	-494.3	-500.2	-509.5	-520.9	-533.7	-547.5	-562.2
40	-63290	-10440	-4297	-2553	-1824	-898.1	-715.0	-650.5	-624.4	-617.1	-618.2	-624.8	-634.7	-649.8	-660.1	-675.8
50	-148600	-22230	-8174	-4380	-2877	-1160	-870.6	-769.8	-726.5	-707.8	-702.0	-703.3	-709.4	-718.7	-730.2	-743.3
60	-306000	-43520	-15060	-7556	-4660	-1528	-1048	-888.5	-817.6	-782.7	-762.6	-758.9	-758.5	-762.3	-769.2	-778.1
70	-567200	-78820	-26350	-12710	-7519	-2065	-1286	-1038	-925.0	-865.5	-832.0	-813.0	-802.8	-798.4	-798.1	-800.6
80	-979400	-133900	-43840	-20620	-11810	-2857	-1633	-1245	-1071	-977.2	-921.3	-885.9	-862.8	-847.7	-838.0	-832.2
90	-1598000	-216100	-69780	-32290	-18250	-3983	-2110	-1529	-1272	-1133	-1048	-992.9	-954.3	-926.5	-906.0	-890.7

VALUES OF K₁ AT ABUTMENT

ϕ_A	025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
10	-100.6	-111.9	-116.0	-118.4	-120.2	-127.9	-135.0	-142.1	-149.2	-156.3	-163.4	-170.4	-177.5	-184.6	-191.7	-198.8
20	-143.2	-165.0	-184.0	-198.6	-206.8	-233.0	-248.0	-262.4	-274.9	-288.7	-300.9	-314.6	-326.7	-340.5	-352.5	-366.3
30	-202.3	-210.3	-222.5	-236.1	-249.2	-296.6	-324.5	-345.5	-364.1	-381.6	-398.7	-415.5	-432.3	-449.0	-465.6	-482.3
40	-268.5	-270.8	-275.9	-282.8	-291.0	-333.7	-367.1	-392.9	-414.6	-434.4	-453.1	-471.3	-489.3	-507.0	-524.7	-542.4
50	-336.9	-337.6	-339.8	-343.1	-347.3	-374.5	-401.6	-424.7	-444.7	-462.7	-479.7	-496.0	-512.0	-527.9	-543.7	-559.5
60	-406.6	-407.2	-408.5	-410.5	-412.9	-429.4	-447.1	-462.9	-476.7	-489.2	-499.3	-512.2	-523.3	-534.5	-545.7	-557.0
70	-447.2	-478.3	-479.8	-481.7	-483.8	-495.9	-507.5	-517.0	-524.4	-530.4	-535.5	-540.3	-545.0	-549.8	-554.7	-559.9
80	-548.8	-550.8	-553.0	-555.3	-557.2	-570.0	-580.2	-587.0	-590.8	-592.4	-592.5	-591.7	-590.6	-589.4	-588.3	-587.3
90	-621.5	-624.5	-627.6	-630.8	-634.0	-649.2	-661.4	-669.5	-673.5	-674.3	-672.6	-669.3	-665.0	-660.1	-654.9	-649.6

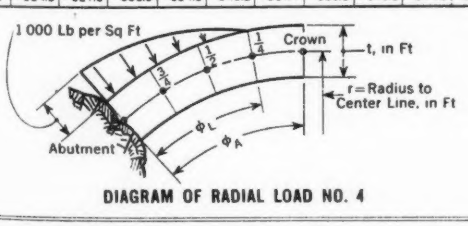


DIAGRAM OF RADIAL LOAD NO. 4

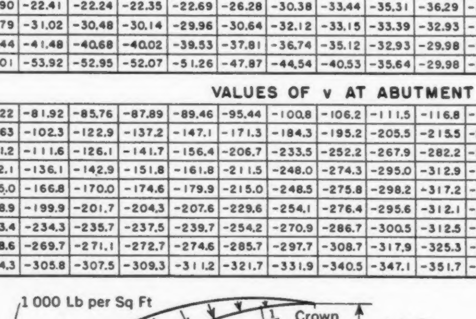
Radial Deflection, in Ft

$$\Delta_r = \frac{K_1 r}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 7(*a*).—Forces and Moments for Radial Load No. 5

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF h AT CROWN																
10	103.2	42.02	26.21	20.22	17.44	14.33	14.49	15.10	15.83	16.58	17.36	18.13	18.91	19.69	20.46	21.24
20	284.5	218.8	164.9	129.3	106.6	67.56	60.99	60.67	62.25	64.57	67.21	70.03	72.93	75.88	78.86	82.11
30	315.7	301.3	278.6	253.9	230.9	162.4	140.7	135.2	135.9	139.3	144.1	149.5	155.4	161.5	167.7	174.1
40	322.4	322.7	318.5	311.3	302.3	257.5	233.3	225.0	225.3	230.0	237.3	245.9	255.4	265.4	275.7	286.3
50	325.6	330.0	332.4	333.1	333.0	320.5	310.2	307.9	312.1	320.7	332.9	345.0	359.1	374.0	389.3	405.0
60	328.5	334.3	339.1	343.0	346.1	355.2	361.3	370.0	382.3	397.6	415.1	434.2	454.2	475.0	496.2	517.8
70	331.9	338.1	343.8	349.0	353.9	374.2	392.3	411.4	432.4	455.5	480.2	506.2	533.2	560.3	588.1	616.2
80	335.8	342.1	348.2	354.1	359.7	385.9	411.3	437.8	466.0	496.0	527.5	560.1	593.5	627.5	661.9	696.5
90	340.3	346.7	353.0	359.1	365.2	394.5	424.1	455.3	488.5	523.7	560.5	598.7	637.9	677.8	718.2	759.0
VALUES OF m AT CROWN																
10	0.921	1.462	1.748	1.967	2.157	2.912	3.511	4.028	4.492	4.923	5.328	5.716	6.091	6.455	6.810	7.160
20	-0.508	1.151	2.674	3.855	4.772	7.651	9.621	11.31	12.86	14.32	15.71	17.06	18.38	19.66	20.92	22.59
30	-2.648	-1.676	-0.230	1.403	3.029	9.370	13.51	16.73	19.54	22.12	24.56	26.92	29.22	31.46	33.66	35.84
40	-5.174	-4.779	-3.989	-2.894	-1.591	5.600	11.59	16.28	20.17	23.59	26.73	29.68	32.52	35.26	37.95	40.58
50	-8.267	-8.235	-7.939	-7.409	-6.682	-1.400	4.291	9.187	12.63	16.71	19.75	22.46	24.98	27.35	29.61	31.80
60	-12.00	-12.23	-12.29	-12.17	-11.93	-9.234	-5.596	-2.201	0.586	2.771	4.463	5.775	6.801	7.612	8.267	8.777
70	-16.42	-16.85	-17.17	-17.39	-17.51	-17.16	-16.12	-15.25	-14.93	-15.24	-16.11	-17.46	-19.26	-21.22	-23.49	-25.95
80	-21.54	-22.16	-22.70	-23.19	-23.61	-25.22	-26.64	-28.52	-31.18	-34.68	-38.94	-43.87	-49.35	-55.28	-62.02	-68.22
90	-27.39	-28.18	-28.94	-29.65	-30.33	-33.58	-37.15	-41.60	-47.20	-54.00	-61.93	-70.86	-80.62	-91.11	-102.2	-113.8
VALUES OF h AT ABUTMENT																
10	106.7	46.58	31.07	25.24	22.56	19.81	20.29	21.21	22.24	23.30	24.38	25.46	26.54	27.63	28.71	29.79
20	287.8	226.3	175.9	142.7	121.6	86.19	81.28	82.24	84.99	88.43	92.18	96.08	100.1	104.1	108.2	112.5
30	319.1	307.2	288.1	267.2	247.8	191.3	175.3	173.4	176.9	182.6	189.6	197.1	205.0	213.1	221.3	229.6
40	327.2	328.4	326.3	321.8	315.8	286.4	272.8	271.5	276.6	285.2	295.7	307.3	319.5	332.1	345.0	358.1
50	333.0	337.4	340.4	342.4	343.8	343.4	344.5	350.6	361.0	374.1	389.6	405.0	421.8	439.0	456.4	474.2
60	339.4	344.5	349.0	353.1	356.9	372.2	386.1	401.3	418.2	436.7	456.3	476.6	497.4	518.6	540.0	561.6
70	347.2	352.2	357.1	361.8	366.3	387.7	408.3	429.3	450.9	473.2	496.8	519.4	543.1	566.8	590.7	614.7
80	356.7	361.5	366.2	370.9	375.5	398.5	421.4	444.4	467.8	491.3	515.3	539.3	563.6	587.8	612.3	636.6
90	367.9	372.5	377.0	381.6	386.1	408.8	431.5	454.2	476.9	499.6	522.4	545.1	567.8	590.5	613.2	635.9
VALUES OF m AT ABUTMENT																
10	-2.644	-3.096	-3.113	-3.048	-2.964	-2.573	-2.288	-2.079	-1.920	-1.796	-1.695	-1.612	-1.543	-1.484	-1.433	-1.389
20	-3.788	-6.344	-8.324	-9.540	-10.24	-10.98	-10.67	-10.26	-9.878	-9.541	-9.247	-8.990	-8.765	-8.566	-8.389	-7.783
30	-5.983	-7.502	-9.659	-11.90	-13.93	-19.58	-21.16	-21.49	-21.41	-21.19	-20.92	-20.65	-20.39	-20.15	-19.92	-19.71
40	-10.02	-10.55	-11.71	-13.30	-15.10	-23.34	-27.97	-30.17	-31.17	-31.59	-31.72	-31.70	-31.61	-31.47	-31.32	-31.16
50	-15.67	-15.58	-15.95	-16.70	-17.56	-24.37	-30.00	-33.56	-36.24	-36.73	-36.97	-37.57	-37.72	-37.59	-37.48	-37.34
60	-22.90	-22.41	-22.24	-22.35	-22.69	-26.28	-30.38	-33.44	-35.31	-36.29	-36.66	-36.66	-36.42	-36.04	-35.59	-35.10
70	-31.79	-31.02	-30.48	-30.14	-29.96	-30.64	-32.12	-33.15	-33.39	-32.93	-31.33	-30.68	-29.15	-27.66	-26.08	-24.51
80	-42.44	-41.48	-40.68	-40.02	-39.53	-37.81	-36.74	-35.12	-32.93	-29.98	-26.72	-23.06	-19.42	-15.62	-12.41	-8.358
90	-55.01	-53.92	-52.95	-52.07	-51.26	-47.87	-44.54	-40.53	-35.64	-29.98	-23.76	-17.21	-10.51	-3.802	2.810	9.260
VALUES OF v AT ABUTMENT																
10	-70.22	-81.92	-85.76	-87.89	-89.46	-95.44	-100.8	-106.2	-111.5	-116.8	-122.1	-127.4	-132.7	-138.0	-143.3	-148.6
20	-77.63	-102.3	-122.9	-137.2	-147.1	-171.3	-184.3	-195.2	-205.5	-215.5	-225.4	-235.2	-245.0	-254.8	-264.6	-274.3
30	-101.2	-111.6	-126.1	-141.7	-156.4	-206.7	-233.5	-252.2	-267.9	-282.2	-295.8	-309.0	-322.1	-335.0	-347.9	-360.7
40	-132.1	-136.1	-142.9	-151.8	-161.8	-211.5	-248.0	-274.3	-295.0	-312.9	-329.2	-344.6	-359.5	-374.0	-388.3	-402.4
50	-165.0	-166.8	-170.0	-174.6	-179.9	-215.0	-248.5	-275.8	-298.2	-317.2	-333.4	-349.7	-364.5	-378.7	-392.5	-406.1
60	-198.9	-199.9	-201.7	-204.3	-207.6	-229.6	-254.1	-276.4	-295.6	-312.1	-326.8	-340.2	-352.7	-364.6	-376.0	-387.2
70	-233.4	-234.3	-235.7	-237.5	-239.7	-254.2	-270.9	-286.7	-300.5	-312.5	-321.1	-332.2	-340.5	-348.6	-356.2	-363.5
80	-268.6	-269.7	-271.1	-272.7	-274.6	-285.7	-297.7	-308.7	-317.9	-325.3	-331.3	-336.2	-340.3	-343.8	-346.9	-349.8
90	-304.3	-305.8	-307.5	-309.3	-311.2	-321.7	-331.9	-340.5	-347.1	-351.7	-354.6	-356.2	-356.8	-356.7	-356.1	-355.1



Thrust, Moment, Shear, in Lb, and Ft-Lb

$H = hr$
$M = mr^2$
$V = vr$

DIAGRAM OF RADIAL LOAD NO. 5

TABLE 7(*b*).—RADIAL DEFLECTIONS FOR RADIAL LOAD No. 5

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF K_1 AT CROWN															
10	- 8417	- 1964	- 912.1	- 583.6	- 442.0	-267.0	-239.6	-234.2	-235.8	-240.5	-246.7	-253.9	-261.6	-269.8	-278.2	-286.9
20	- 15690	- 8320	- 4640	- 2936	- 2061	-842.2	-625.3	-557.6	-533.3	-526.8	-529.1	-536.2	-546.3	-558.4	-569.0	-585.8
30	- 10020	- 7305	- 6198	- 4794	- 3764	-1662	-1137	-948.5	-870.7	-833.7	-821.3	-820.0	-826.3	-838.0	-852.6	-869.6
40	83040	2256	- 3662	- 4163	- 3857	-2243	-1587	-1307	-1174	-1108	-1077	-1066	-1066	-1075	-1088	-1106
50	238200	23130	2931	- 1283	- 2187	-2297	-1788	-1514	-1385	-1290	-1246	-1232	-1230	-1236	-1249	-1267
60	525400	61380	15060	4197	587.7	-1865	-1700	-1520	-1407	-1343	-1309	-1295	-1294	-1302	-1317	-1336
70	1011000	125500	35240	13270	5551	-1001	-1421	-1342	-1296	-1265	-1250	-1248	-1255	-1269	-1289	-1312
80	1781000	226700	66860	27380	21480	335.5	-797.5	-1005	-1055	-1073	-1087	-1103	-1122	-1145	-1173	-1198
90	2949000	379400	114300	48430	24570	2260	7.967	-518.5	-699.4	-783.3	-834.5	-873.6	-910.0	-940.9	-988.3	-1006
	VALUES OF K_1 AT $\frac{1}{4}$ POINT															
10	- 7736	- 1827	- 862.1	- 558.9	- 427.5	-263.7	-238.0	-233.1	-235.0	-239.8	-246.1	-253.3	-261.1	-269.2	-277.7	-286.4
20	- 16810	- 7876	- 4346	- 2756	- 1945	-816.4	-614.4	-550.8	-528.4	-522.8	-525.6	-533.0	-543.5	-555.5	-566.8	-583.1
30	- 5330	- 8488	- 6219	- 4644	- 3604	-1598	-1106	-930.5	-859.3	-823.6	-812.1	-812.1	-819.2	-830.8	-856.0	-862.6
40	28840	- 4057	- 5242	- 4656	- 4002	-2183	-1547	-1281	-1155	-1093	-1064	-1053	-1054	-1063	-1076	-1094
50	101000	5855	- 2078	- 3296	- 2232	-2343	-1738	-1499	-1356	-1277	-1231	-1220	-1216	-1223	-1235	-1252
60	233500	23860	3744	- 6205	- 3500	-2153	-1851	-1549	-1421	-1349	-1310	-1293	-1290	-1296	-1309	-1326
70	456400	53760	13310	3759	5574	-1700	-1597	-1457	-1366	-1313	-1285	-1274	-1274	-1283	-1298	-1316
80	808100	100600	28150	10480	4269	-998.5	-1280	-1259	-1218	-1191	-1178	-1176	-1182	-1195	-1214	-1232
90	1338000	170700	50200	20390	9686	-10.23	-835.0	-972.3	-997.5	-1003	-1008	-1017	-1030	-1047	-1080	-1087
	VALUES OF K_1 AT $\frac{1}{2}$ POINT															
10	- 5676	- 1410	- 703.7	- 477.1	- 377.1	-249.9	-230.2	-227.6	-230.5	-235.9	-239.9	-250.1	-258.1	-266.4	-274.9	-283.7
20	- 16330	- 6193	- 3375	- 2176	- 1571	-723.7	-568.9	-521.2	-505.9	-504.0	-509.0	-517.8	-529.1	-542.1	-554.4	-570.3
30	- 28570	- 9291	- 5509	- 3875	- 2958	-1364	-988.9	-855.4	-803.6	-777.9	-772.1	-775.8	-785.2	-798.3	-814.0	-831.6
40	- 59350	- 13520	- 7138	- 4917	- 2638	-1891	-1323	-1161	-1053	-1018	-993.8	-992.9	-995.5	-1008	-1022	-1040
50	-125400	- 22070	- 9809	- 6157	- 4450	-2208	-1622	-1374	-1259	-1189	-1448	-1146	-1145	-1152	-1165	-1181
60	-248700	- 37980	-14670	- 8276	- 5673	-2439	-1768	-1494	-1358	-1285	-1247	-1229	-1224	-1228	-1238	-1253
70	-458900	- 65050	-22920	-11860	- 7556	-2717	-1878	-1562	-1408	-1325	-1280	-1257	-1247	-1248	-1254	-1266
80	-794400	-108300	-36100	-17570	-10560	-3138	-2014	-1622	-1437	-1338	-1282	-1251	-1236	-1230	-1233	-1239
90	-1306000	-174000	-56150	-26280	-15150	-3785	-2220	-1709	-1475	-1350	-1278	-1235	-1210	-1197	-1203	-1193
	VALUES OF K_1 AT $\frac{3}{4}$ POINT															
10	- 2632	- 775.1	- 447.1	- 336.3	- 285.7	-220.1	-212.0	-214.1	-219.4	-226.3	-234.0	-242.1	-250.6	-259.4	-268.2	-277.2
20	- 9142	- 3128	- 1785	- 1234	- 953.8	-550.3	-476.0	-456.6	-454.8	-460.7	-470.5	-482.6	-496.1	-510.6	-525.0	-541.2
30	- 23980	- 5792	- 3136	- 2186	-1704.8	-932.8	-757.1	-698.3	-682.1	-676.0	-682.2	-693.6	-708.2	-725.0	-742.9	-762.7
40	- 62370	- 11230	- 5043	- 3197	- 2386	-1250	-993.8	-898.6	-859.6	-846.1	-827.2	-853.9	-866.7	-882.9	-901.4	-921.6
50	-143100	- 22320	- 8665	- 4898	- 3336	-1521	-1172	-1043	-989.0	-962.2	-952.7	-957.4	-966.5	-979.8	-996.2	-1015
60	-291500	- 42420	-15130	- 7859	- 5018	-1852	-1337	-1158	-1078	-1039	-1022	-1017	-1020	-1028	-1040	-1055
70	-540500	- 75850	-25770	-12690	- 7683	-2338	-1541	-1285	-1166	-1106	-1074	-1058	-1051	-1052	-1057	-1065
80	-932700	-128100	-42310	-20150	-11770	-3058	-1847	-1457	-1283	-1190	-1137	-1105	-1086	-1076	-1071	-1071
90	-1522000	-206300	-66910	-31180	-17780	-4095	-2271	-1701	-1448	-1312	-1231	-1179	-1145	-1121	-1111	-1095
	VALUES OF K_1 AT ABUTMENT															
10	-125.3	-146.2	-153.1	-156.9	-159.7	-170.4	-180.0	-189.5	-199.0	-208.5	-218.0	-227.4	-238.9	-246.4	-255.9	-265.3
20	-138.6	-182.5	-219.3	-244.9	-262.6	-305.7	-329.0	-348.4	-366.8	-384.6	-402.3	-419.8	-437.3	-454.8	-472.3	-489.6
30	-180.7	-199.2	-225.2	-253.0	-278.5	-368.9	-415.8	-450.2	-480.5	-503.7	-526.6	-551.6	-573.5	-589.1	-619.5	-643.9
40	-235.8	-242.9	-255.1	-270.9	-288.7	-377.5	-442.7	-489.6	-526.6	-558.5	-587.7	-615.1	-641.7	-667.6	-693.1	-718.3
50	-294.6	-297.6	-303.5	-311.7	-321.1	-383.8	-443.5	-492.3	-532.2	-566.2	-595.1	-624.2	-650.6	-675.9	-700.6	-724.9
60	-355.1	-356.8	-360.1	-364.7	-370.5	-409.8	-453.5	-493.3	-527.6	-557.2	-583.4	-607.3	-629.5	-650.7	-671.2	-691.1
70	-416.7	-418.3	-420.8	-424.0	-427.9	-453.8	-483.6	-511.7	-536.4	-557.7	-576.4	-593.0	-607.8	-622.3	-635.8	-648.8
80	-479.4	-481.4	-483.9	-486.8	-490.1	-510.0	-531.4	-551.0	-567.3	-580.6	-591.3	-600.1	-607.3	-613.6	-619.1	-624.4
90	-543.1	-545.9	-548.9	-552.1	-555.6	-574.2	-592.4	-607.8	-619.5	-627.7	-633.0	-635.8	-636.9	-636.7	-635.6	-633.9

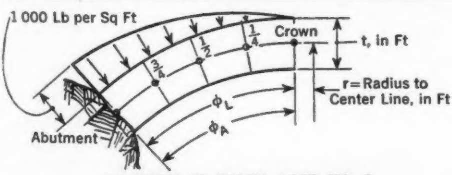


DIAGRAM OF RADIAL LOAD NO. 5

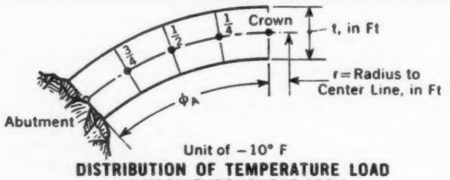
Radial Deflection, in Ft

$$\Delta_r = \frac{K_1 r}{E_C}$$

E_C = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 8(*a*).—Forces and Moments for Temperature Load

ϕ_A	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF t/r															
	VALUES OF h AT CROWN															
10	$-.0^6710$	$-.0^5154$	$-.0^5211$	$-.0^5253$	$-.0^5286$	$-.0^5380$	$-.0^5426$	$-.0^5452$	$-.0^5470$	$-.0^5482$	$-.0^5491$	$-.0^5499$	$-.0^5504$	$-.0^5509$	$-.0^5513$	$-.0^5516$
20	$-.0^6147$	$-.0^6741$	$-.0^5153$	$-.0^5230$	$-.0^5298$	$-.0^5522$	$-.0^5645$	$-.0^5722$	$-.0^5776$	$-.0^5816$	$-.0^5846$	$-.0^5870$	$-.0^5890$	$-.0^5906$	$-.0^5920$	$-.0^5932$
30	$-.0^7340$	$-.0^6227$	$-.0^6618$	$-.0^5116$	$-.0^5178$	$-.0^5476$	$-.0^5683$	$-.0^5823$	$-.0^5924$	$-.0^4100$	$-.0^4106$	$-.0^4111$	$-.0^4115$	$-.0^4118$	$-.0^4121$	$-.0^4123$
40	$-.0^7112$	$-.0^7810$	$-.0^6243$	$-.0^6507$	$-.0^6863$	$-.0^5335$	$-.0^5579$	$-.0^5772$	$-.0^5921$	$-.0^4104$	$-.0^4113$	$-.0^4120$	$-.0^4127$	$-.0^4132$	$-.0^4136$	$-.0^4140$
50	$-.0^8465$	$-.0^7347$	$-.0^6108$	$-.0^6236$	$-.0^6423$	$-.0^5208$	$-.0^5425$	$-.0^5631$	$-.0^5808$	$-.0^5954$	$-.0^4108$	$-.0^4118$	$-.0^4126$	$-.0^4134$	$-.0^4140$	$-.0^4145$
60	$-.0^8224$	$-.0^7170$	$-.0^7543$	$-.0^6121$	$-.0^6222$	$-.0^5125$	$-.0^5289$	$-.0^5473$	$-.0^5649$	$-.0^5807$	$-.0^5946$	$-.0^4107$	$-.0^4117$	$-.0^4127$	$-.0^4134$	$-.0^4141$
70	$-.0^8120$	$-.0^8922$	$-.0^7298$	$-.0^7673$	$-.0^6125$	$-.0^6766$	$-.0^5193$	$-.0^5339$	$-.0^5495$	$-.0^5645$	$-.0^5785$	$-.0^5912$	$-.0^4103$	$-.0^4113$	$-.0^4122$	$-.0^4130$
80	$-.0^9696$	$-.0^8537$	$-.0^7175$	$-.0^7398$	$-.0^7748$	$-.0^6483$	$-.0^5129$	$-.0^5240$	$-.0^5366$	$-.0^5498$	$-.0^5627$	$-.0^5750$	$-.0^5863$	$-.0^5968$	$-.0^4106$	$-.0^4115$
90	$-.0^9425$	$-.0^8330$	$-.0^7108$	$-.0^7248$	$-.0^7468$	$-.0^6314$	$-.0^6873$	$-.0^5169$	$-.0^5268$	$-.0^5377$	$-.0^5489$	$-.0^5600$	$-.0^5706$	$-.0^5807$	$-.0^5901$	$-.0^5988$
	VALUES OF m AT CROWN															
10	$.0^8401$	$.0^8946$	$.0^7140$	$.0^7178$	$.0^7212$	$.0^7338$	$.0^7421$	$.0^7480$	$.0^7525$	$.0^7559$	$.0^7587$	$.0^7610$	$.0^7629$	$.0^7646$	$.0^7659$	$.0^7672$
20	$.0^8314$	$.0^7167$	$.0^7361$	$.0^7564$	$.0^7758$	$.0^6154$	$.0^6211$	$.0^6225$	$.0^6291$	$.0^6321$	$.0^6346$	$.0^6367$	$.0^6386$	$.0^6402$	$.0^6416$	$.0^6429$
30	$.0^8159$	$.0^7110$	$.0^7310$	$.0^7600$	$.0^7949$	$.0^6286$	$.0^6449$	$.0^6582$	$.0^6692$	$.0^6786$	$.0^6867$	$.0^6938$	$.0^5100$	$.0^5106$	$.0^5111$	$.0^5115$
40	$.0^9914$	$.0^8679$	$.0^7209$	$.0^7447$	$.0^7779$	$.0^6334$	$.0^6625$	$.0^6890$	$.0^5112$	$.0^5132$	$.0^5150$	$.0^5166$	$.0^5180$	$.0^5192$	$.0^5204$	$.0^5214$
50	$.0^9581$	$.0^8443$	$.0^7141$	$.0^7315$	$.0^7573$	$.0^6306$	$.0^6673$	$.0^5106$	$.0^5143$	$.0^5176$	$.0^5207$	$.0^5234$	$.0^5259$	$.0^5282$	$.0^5302$	$.0^5322$
60	$.0^9396$	$.0^8306$	$.0^8991$	$.0^7225$	$.0^7419$	$.0^6254$	$.0^6626$	$.0^5108$	$.0^5155$	$.0^5201$	$.0^5244$	$.0^5285$	$.0^5322$	$.0^5356$	$.0^5388$	$.0^5418$
70	$.0^9282$	$.0^8220$	$.0^8719$	$.0^7165$	$.0^7311$	$.0^6203$	$.0^6540$	$.0^6997$	$.0^5152$	$.0^5206$	$.0^5259$	$.0^5311$	$.0^5359$	$.0^5406$	$.0^5449$	$.0^5489$
80	$.0^9208$	$.0^8163$	$.0^8535$	$.0^7124$	$.0^7235$	$.0^6160$	$.0^6450$	$.0^6874$	$.0^5139$	$.0^5196$	$.0^5255$	$.0^5314$	$.0^5371$	$.0^5426$	$.0^5480$	$.0^5530$
90	$.0^9156$	$.0^8123$	$.0^8406$	$.0^8941$	$.0^7180$	$.0^6127$	$.0^6368$	$.0^6741$	$.0^5122$	$.0^5178$	$.0^5237$	$.0^5299$	$.0^5361$	$.0^5423$	$.0^5482$	$.0^5540$
	VALUES OF h AT ABUTMENT															
10	$-.0^6700$	$-.0^5151$	$-.0^5208$	$-.0^5249$	$-.0^5282$	$-.0^5375$	$-.0^5419$	$-.0^5445$	$-.0^5463$	$-.0^5475$	$-.0^5484$	$-.0^5491$	$-.0^5497$	$-.0^5501$	$-.0^5505$	$-.0^5508$
20	$-.0^6138$	$-.0^6697$	$-.0^5144$	$-.0^5216$	$-.0^5280$	$-.0^5490$	$-.0^5606$	$-.0^5679$	$-.0^5729$	$-.0^5767$	$-.0^5795$	$-.0^5818$	$-.0^5836$	$-.0^5852$	$-.0^5864$	$-.0^5875$
30	$-.0^7295$	$-.0^6197$	$-.0^6535$	$-.0^5100$	$-.0^5154$	$-.0^5412$	$-.0^5591$	$-.0^5713$	$-.0^5800$	$-.0^5866$	$-.0^5917$	$-.0^5958$	$-.0^5992$	$-.0^4102$	$-.0^4104$	$-.0^4106$
40	$-.0^8858$	$-.0^7620$	$-.0^6186$	$-.0^6388$	$-.0^6661$	$-.0^5257$	$-.0^5444$	$-.0^5592$	$-.0^5705$	$-.0^5794$	$-.0^5864$	$-.0^5922$	$-.0^5969$	$-.0^4101$	$-.0^4104$	$-.0^4107$
50	$-.0^8299$	$-.0^7223$	$-.0^7697$	$-.0^6152$	$-.0^6272$	$-.0^5133$	$-.0^5273$	$-.0^5406$	$-.0^5519$	$-.0^5613$	$-.0^5691$	$-.0^5757$	$-.0^5812$	$-.0^5859$	$-.0^5899$	$-.0^5935$
60	$-.0^8112$	$-.0^8851$	$-.0^7271$	$-.0^7606$	$-.0^6111$	$-.0^6625$	$-.0^5145$	$-.0^5237$	$-.0^5325$	$-.0^5404$	$-.0^5473$	$-.0^5533$	$-.0^5585$	$-.0^5630$	$-.0^5670$	$-.0^5705$
70	$-.0^9412$	$-.0^8316$	$-.0^7102$	$-.0^7230$	$-.0^7428$	$-.0^6262$	$-.0^6660$	$-.0^5116$	$-.0^5169$	$-.0^5221$	$-.0^5270$	$-.0^5312$	$-.0^5351$	$-.0^5385$	$-.0^5416$	$-.0^5444$
80	$-.0^9121$	$-.0^9933$	$-.0^8303$	$-.0^8692$	$-.0^7130$	$-.0^7838$	$-.0^6224$	$-.0^6416$	$-.0^6636$	$-.0^6865$	$-.0^5109$	$-.0^5130$	$-.0^5150$	$-.0^5168$	$-.0^5185$	$-.0^5200$
90	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	VALUES OF m AT ABUTMENT															
10	$-.0^8678$	$-.0^7139$	$-.0^7181$	$-.0^7206$	$-.0^7222$	$-.0^7240$	$-.0^7226$	$-.0^7207$	$-.0^7189$	$-.0^7173$	$-.0^7159$	$-.0^7147$	$-.0^7137$	$-.0^7128$	$-.0^7120$	$-.0^7113$
20	$-.0^8573$	$-.0^7280$	$-.0^7564$	$-.0^7823$	$-.0^6104$	$-.0^6161$	$-.0^6178$	$-.0^6180$	$-.0^6177$	$-.0^6172$	$-.0^6165$	$-.0^6158$	$-.0^6151$	$-.0^6145$	$-.0^6138$	$-.0^6133$
30	$-.0^8296$	$-.0^7194$	$-.0^7518$	$-.0^7954$	$-.0^6144$	$-.0^6352$	$-.0^6466$	$-.0^6521$	$-.0^6546$	$-.0^6553$	$-.0^6552$	$-.0^6544$	$-.0^6533$	$-.0^6521$	$-.0^6508$	$-.0^6494$
40	$-.0^8171$	$-.0^7122$	$-.0^7360$	$-.0^7740$	$-.0^6124$	$-.0^6450$	$-.0^6730$	$-.0^6917$	$-.0^5103$	$-.0^5110$	$-.0^5114$	$-.0^5116$	$-.0^5116$	$-.0^5116$	$-.0^5115$	$-.0^5114$
50	$-.0^8108$	$-.0^8796$	$-.0^7246$	$-.0^7530$	$-.0^7937$	$-.0^6435$	$-.0^6845$	$-.0^5119$	$-.0^5146$	$-.0^5165$	$-.0^5178$	$-.0^5186$	$-.0^5192$	$-.0^5196$	$-.0^5197$	$-.0^5198$
60	$-.0^9727$	$-.0^8546$	$-.0^7172$	$-.0^7381$	$-.0^7691$	$-.0^6371$	$-.0^6822$	$-.0^5129$	$-.0^5169$	$-.0^5203$	$-.0^5229$	$-.0^5248$	$-.0^5263$	$-.0^5274$	$-.0^5282$	$-.0^5287$
70	$-.0^9510$	$-.0^8387$	$-.0^7124$	$-.0^7278$	$-.0^7512$	$-.0^6301$	$-.0^6730$	$-.0^5124$	$-.0^5174$	$-.0^5219$	$-.0^5258$	$-.0^5289$	$-.0^5315$	$-.0^5336$	$-.0^5352$	$-.0^5365$
80	$-.0^9367$	$-.0^8281$	$-.0^8908$	$-.0^7206$	$-.0^7383$	$-.0^6239$	$-.0^6616$	$-.0^5111$	$-.0^5164$	$-.0^5216$	$-.0^5264$	$-.0^5306$	$-.0^5342$	$-.0^5373$	$-.0^5399$	$-.0^5421$
90	$-.0^9269$	$-.0^8207$	$-.0^8673$	$-.0^7154$	$-.0^7288$	$-.0^6187$	$-.0^6505$	$-.0^6948$	$-.0^5146$	$-.0^5200$	$-.0^5252$	$-.0^5301$	$-.0^5345$	$-.0^5384$	$-.0^5419$	$-.0^5448$
	VALUES OF v AT ABUTMENT															
10	$-.0^6123$	$-.0^6267$	$-.0^6366$	$-.0^6440$	$-.0^6497$	$-.0^6660$	$-.0^6739$	$-.0^6785$	$-.0^6816$	$-.0^6837$	$-.0^6853$	$-.0^6866$	$-.0^6876$	$-.0^6884$	$-.0^6891$	$-.0^6897$
20	$-.0^7503$	$-.0^6254$	$-.0^6524$	$-.0^6786$	$-.0^5102$	$-.0^5178$	$-.0^5207$	$-.0^5247$	$-.0^5265$	$-.0^5279$	$-.0^5289$	$-.0^5298$	$-.0^5304$	$-.0^5310$	$-.0^5315$	$-.0^5319$
30	$-.0^7170$	$-.0^6114$	$-.0^6309$	$-.0^6580$	$-.0^6891$	$-.0^5238$	$-.0^5341$	$-.0^5412$	$-.0^5462$	$-.0^5500$	$-.0^5529$	$-.0^5553$	$-.0^5573$	$-.0^5589$	$-.0^5603$	$-.0^5615$
40	$-.0^8720$	$-.0^7521$	$-.0^6156$	$-.0^6326$	$-.0^6555$	$-.0^5215$	$-.0^5372$	$-.0^5496$	$-.0^5592$	$-.0^5666$	$-.0^5725$	$-.0^5773$	$-.0^5813$	$-.0^5847$	$-.0^5876$	$-.0^5901$
50	$-.0^8356$	$-.0^7266$	$-.0^7830$	$-.0^6181$	$-.0^6324$	$-.0^5159$	$-.0^5326$	$-.0^5484$	$-.0^5619$	$-.0^5731$	$-.0^5824$	$-.0^5902$	$-.0^5968$	$-.0^4102$	$-.0^4107$	$-.0^4111$
60	$-.0^8194$	$-.0^7147$	$-.0^7470$	$-.0^6105$	$-.0^6192$	$-.0^5108$	$-.0^5251$	$-.0^5410$	$-.0^5562$	$-.0^5699$	$-.0^5819$	$-.0^5923$	$-.0^4101$	$-.0^4110$	$-.0^4116$	$-.0^4122$
70	$-.0^8113$	$-.0^8867$	$-.0^7280$	$-.0^7632$	$-.0^6118$	$-.0^6720$	$-.0^5181$	$-.0^5319$	$-.0^5465$	$-.0^5606$	$-.0^5741$	$-.0^5857$	$-.0^5964$	$-.0^4106$	$-.0^4114$	$-.0^4122$
80	$-.0^9685$	$-.0^8529$	$-.0^7172$	$-.0^7392$	$-.0^7737$	$-.0^6476$	$-.0^5127$	$-.0^5236$	$-.0^5361$	$-.0^5491$	$-.0^5618$	$-.0^5738$	$-.0^5850$	$-.0^5953$	$-.0^4105$	$-.0^4113$
90	$-.0^9425$	$-.0^8330$	$-.0^7108$	$-.0^7248$	$-.0^7468$	$-.0^6314$	$-.0^6873$	$-.0^5169$	$-.0^5268$	$-.0^5377$	$-.0^5489$	$-.0^5600$	$-.0^5706$	$-.0^5807$	$-.0^5901$	$-.0^5988$



Unit of −10° F

DISTRIBUTION OF TEMPERATURE LOAD

UNIFORM THROUGHOUT ARCH

Thrust, Moment, Shear in Lb, and Ft-Lb

$H = h\,r\,E_c$

$M = m\,r^2\,E_c$

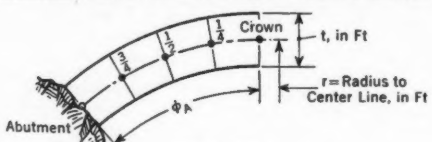
$V = v\,r\,E_c$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

TABLE 8(*b*).—Radial Deflections for Temperature Load

ϕ_A	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
VALUES OF K_1 AT CROWN																
10	-5.47	-1.97	-1.08	-.69	-0.43	-0.33	-0.28	-0.25	-0.24	-0.23	-.225	-0.22	-0.22	-0.22	-.215	-0.21
20	-1 590	-1 145	-8.07	-5.91	-4.52	-2.09	-1.56	-1.29	-1.13	-1.06	-1.01	-0.97	-0.95	-0.93	-9.15	0.90
30	-1 793	-1 630	-1 437	-1 248	-1 049	-5.93	-4.22	-3.31	-2.85	-2.62	-2.45	-2.33	-2.23	-2.17	-2.12	-2.07
40	-1 841	-1 771	-1 682	-1 582	-1 480	-1 031	-7.71	-6.24	-5.39	-4.85	-4.48	-4.22	-4.03	-3.88	-3.78	-3.67
50	-1 861	-1 823	-1 776	-1 722	-1 667	-1 354	-1 105	-9.37	-8.22	-7.43	-6.87	-6.46	-6.15	-5.91	-5.72	-5.56
60	-1 875	-1 850	-1 821	-1 788	-1 753	-1 552	-1 360	-1 204	-1 086	-9.96	-9.29	-8.77	-8.37	-8.05	-7.79	-7.58
70	-1 886	-1 868	-1 848	-1 826	-1 802	-1 669	-1 532	-1 405	-1 300	-1 215	-1 147	-1 091	-1 047	-1 011	-9.82	-9.56
80	-1 897	-1 883	-1 868	-1 853	-1 836	-1 743	-1 643	-1 548	-1 463	-1 390	-1 328	-1 276	-1 232	-1 196	-1 166	-1 139
90	-1 908	-1 897	-1 886	-1 874	-1 862	-1 794	-1 722	-1 650	-1 584	-1 525	-1 474	-1 428	-1 388	-1 355	-1 327	-1 301
VALUES OF K_1 AT $\frac{1}{4}$ POINT																
10	-4.88	-1.78	-0.99	-0.69	-0.52	-0.32	-0.28	-0.25	-0.24	-0.24	-0.23	-0.23	-0.22	-0.22	-0.22	-0.22
20	-1 408	-1 021	-0 725	-5.35	-4.17	-1.97	-1.46	-1.25	-1.14	-1.05	-1.00	-0.98	-0.96	-0.94	-.925	-0.91
30	-1 582	-1 446	-1 281	-1 118	-9.61	-5.47	-3.86	-3.16	-2.78	-2.55	-2.41	-2.30	-2.22	-2.16	-2.12	-2.08
40	-1 621	-1 565	-1 492	-1 408	-1 320	-9.38	-7.15	-5.86	-5.11	-4.65	-4.33	-4.10	-3.94	-3.81	-3.72	-3.63
50	-1 636	-1 607	-1 570	-1 526	-1 479	-1 222	-1 010	-8.67	-7.70	-7.02	-6.54	-6.19	-5.94	-5.73	-5.57	-5.43
60	-1 644	-1 626	-1 605	-1 580	-1 552	-1 390	-1 232	-1 103	-1 004	-9.29	-8.73	-8.30	-7.97	-7.70	-7.48	-7.31
70	-1 651	-1 638	-1 624	-1 608	-1 590	-1 487	-1 377	-1 275	-1 189	-1 120	-1 064	-1 020	-9.84	-9.55	-9.31	-9.10
80	-1 657	-1 647	-1 637	-1 626	-1 614	-1 545	-1 469	-1 394	-1 327	-1 269	-1 220	-1 179	-1 145	-1 116	-1 091	-1 072
90	-1 662	-1 655	-1 648	-1 640	-1 631	-1 582	-1 529	-1 475	-1 425	-1 380	-1 340	-1 306	-1 276	-1 251	-1 229	-1 210
VALUES OF K_1 AT $\frac{1}{2}$ POINT																
10	-3.32	-1.29	-.756	-.549	-0.45	-.302	-0.28	-0.26	-0.25	-0.25	-0.24	-0.24	-0.24	-0.24	-0.24	-0.24
20	-9.28	-6.93	-5.07	-3.85	-3.10	-1.63	-1.29	-1.14	-1.08	-1.04	-1.01	-0.994	-0.98	-.972	-0.96	-.958
30	-1 030	-9.60	-8.68	-7.72	-6.85	-4.20	-3.23	-2.74	-2.49	-2.36	-2.27	-2.22	-2.17	-2.14	-2.12	-2.10
40	-1 047	-1 026	-9.92	-9.50	-9.02	-6.88	-5.58	-4.79	-4.33	-4.08	-3.90	-3.77	-3.68	-3.62	-3.56	-3.52
50	-1 049	-1 043	-1 031	-1 014	-9.90	-8.67	-7.58	-6.77	-6.23	-5.88	-5.62	-5.44	-5.31	-5.21	-5.13	-5.07
60	-1 048	-1 047	-1 043	-1 036	-1 026	-9.62	-8.92	-8.28	-7.80	-7.45	-7.18	-6.99	-6.83	-6.72	-6.62	-6.55
70	-1 046	-1 046	-1 045	-1 043	-1 039	-1 008	-9.68	-9.27	-8.92	-8.64	-8.43	-8.25	-8.11	-8.00	-7.92	-7.85
80	-1 042	-1 043	-1 044	-1 043	-1 042	-1 029	-1 009	-9.86	-9.65	-9.46	-9.30	-9.18	-9.08	-9.00	-8.93	-8.88
90	-1 038	-1 040	-1 040	-1 041	-1 041	-1 037	-1 028	-1 017	-1 006	-9.96	-9.87	-9.81	-9.76	-9.71	-9.68	-9.65
VALUES OF K_1 AT $\frac{3}{4}$ POINT																
10	-1.38	-.633	-.432	-.353	-0.30	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27	-0.27
20	-3.50	-2.87	-2.29	-1.89	-1.62	-1.15	-1.07	-1.02	-1.01	-1.01	-1.01	-1.02	-1.02	-1.03	-1.03	-1.03
30	-3.74	-3.72	-3.57	-3.37	-3.17	-2.47	-2.20	-2.13	-2.10	-2.09	-2.09	-2.10	-2.10	-2.11	-2.12	-2.13
40	-3.72	-3.83	-3.88	-3.88	-3.84	-3.58	-3.39	-3.30	-3.26	-3.26	-3.27	-3.29	-3.31	-3.33	-3.35	-3.37
50	-3.67	-3.79	-3.89	-3.96	-4.01	-4.13	-4.17	-4.18	-4.21	-4.27	-4.32	-4.37	-4.41	-4.46	-4.50	-4.54
60	-3.61	-3.72	-3.82	-3.91	-3.99	-4.29	-4.50	-4.66	-4.80	-4.93	-5.05	-5.15	-5.24	-5.33	-5.40	-5.48
70	-3.55	-3.65	-3.74	-3.83	-3.91	-4.26	-4.55	-4.81	-5.04	-5.25	-5.44	-5.60	-5.74	-5.88	-6.00	-6.11
80	-3.49	-3.57	-3.65	-3.73	-3.81	-4.15	-4.46	-4.76	-5.03	-5.29	-5.52	-5.73	-5.92	-6.10	-6.26	-6.41
90	-3.43	-3.50	-3.57	-3.63	-3.69	-4.01	-4.31	-4.60	-4.87	-5.15	-5.41	-5.65	-5.87	-6.07	-6.25	-6.43
VALUES OF K_1 AT ABUTMENT																
10	-.0 440	-.0 952	-.131	-.157	-.177	-.236	-.264	-.280	-.291	-.299	-0.304	-0.309	-.313	-.316	-.318	-.32
20	-.0 180	-.0 905	-.187	-.281	-.364	-.637	-.739	-.882	-.946	-.996	-1.03	-1.06	-1.09	-1.11	-1.12	-1.14
30	$-.0^{2}607$	-.0 406	-.110	-.207	-.318	-.850	-1.22	-1.47	-1.65	-1.78	-1.89	-1.97	-2.05	-2.10	-2.15	-2.19
40	$-.0^{2}257$	-.0 186	-.0 558	-.116	-.198	-.769	-1.33	-1.77	-2.11	-2.38	-2.59	-2.76	-2.90	-3.05	-3.13	-3.22
50	$-.0^{2}127$	$-.0^{2}949$	-.0 296	-.0 647	-.116	-.568	-1.16	-1.73	-2.21	-2.61	-2.94	-3.22	-3.46	-3.65	-3.82	-3.98
60	$-.0^{3}694$	$-.0^{2}526$	-.0 168	-.0 375	-.0 685	-.387	-.896	-1.46	-2.01	-2.50	-2.92	-3.30	-3.61	-3.90	-4.14	-4.36
70	$-.0^{3}404$	$-.0^{2}309$	$-.0^{2}999$	-.0 226	-.0 421	-.257	-.646	-1.14	-1.66	-2.16	-2.65	-3.06	-3.44	-3.78	-4.07	-4.36
80	$-.0^{3}245$	$-.0^{2}189$	$-.0^{2}614$	-.0 140	-.0 263	-.170	-.453	-.842	-1.29	-1.75	-2.21	-2.64	-3.03	-3.40	-3.75	-4.04
90	$-.0^{3}152$	$-.0^{2}118$	$-.0^{2}385$	$-.0^{2}884$	.0 167	-.112	-.312	-.603	-.957	-1.35	-1.75	-2.14	-2.52	-2.88	-3.22	-3.53

(Column heading: VALUES OF t/r)



Unit of −10° F

DISTRIBUTION OF TEMPERATURE LOAD

UNIFORM THROUGHOUT ARCH

Radial Deflection, in Ft

$\Delta_r = K_1 r c T$

C = Coefficient of Thermal Expansion of Concrete per Degree Fahrenheit Change in Temperature

T = Temperature Change in Degrees Fahrenheit ÷ (−10)

Example (Δ_r at Crown):

$\phi_A = 10°$; $\frac{t}{r} = 0.125$; T = 25° F

$\Delta_r = (-0.43)\,(r)\,(c)\,(-2.5)$

TABLE 9(*a*).—Forces and Moments for Concentrated Radial Load at Abutment

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	1.000	1.125	1.250	1.375	1.500
	VALUES OF h AT CROWN															
10	25.36	54.83	75.29	90.41	102.1	135.8	151.9	161.4	167.7	172.1	175.4	178.0	180.1	181.7	183.1	184.3
20	5.251	26.47	54.72	82.09	106.2	186.3	230.1	257.9	276.5	291.3	302.1	310.8	317.7	323.5	328.4	332.6
30	1.214	8.110	22.07	41.40	63.64	170.0	243.8	293.9	329.9	356.9	378.0	395.0	408.9	420.5	430.4	438.8
40	.4000	2.891	8.685	18.10	30.83	119.6	206.9	275.7	328.6	369.9	402.8	429.5	451.7	470.4	486.4	500.2
50	.1660	1.238	3.870	8.443	15.10	74.13	151.7	225.4	236.6	340.7	384.0	420.3	450.9	477.0	499.6	519.2
60	.08014	.6079	1.938	4.325	7.929	44.65	103.4	168.9	231.8	288.2	337.6	380.5	417.7	450.1	478.4	503.4
70	.04297	.3293	1.063	2.403	4.469	27.34	68.89	121.2	176.5	230.3	281.3	325.5	366.1	402.3	434.7	463.7
80	.02484	.1918	.6237	1.423	2.671	17.24	46.03	85.53	130.8	177.8	223.9	267.6	308.2	345.5	379.6	410.6
90	.01517	.1178	.3852	.8842	1.671	11.19	31.15	60.30	95.81	134.7	174.7	214.2	252.2	288.0	321.6	352.8
	VALUES OF m AT CROWN															
10	-.1432	-.3379	-.4988	-.6368	-.7581	-1.207	-1.502	-1.713	-1.873	-1.997	-2.096	-2.178	-2.246	-2.304	-2.354	-2.398
20	-.1122	-.5949	-1.287	-2.013	-2.705	-5.498	-7.529	-9.108	-10.38	-11.44	-12.34	-13.10	-13.77	-14.35	-14.86	-15.32
30	-.05687	-.3935	-1.123	-2.141	-3.387	-10.20	-16.02	-20.76	-24.71	-28.06	-30.95	-33.49	-35.74	-37.73	-39.53	-41.15
40	-.03263	-.2424	-.7469	-1.595	-2.779	-11.92	-22.32	-31.77	-40.01	-47.20	-53.51	-59.11	-64.13	-68.65	-72.75	-76.49
50	-.02074	-.1582	-.5048	-1.123	-2.047	-10.94	-24.02	-37.87	-41.83	-62.98	-73.79	-83.60	-92.49	-100.6	-108.0	-114.8
60	-.01413	-.1091	-.3540	-.8032	-1.496	-9.064	-22.34	-38.50	-55.37	-71.77	-87.24	-101.6	-114.9	-127.2	-138.6	-149.2
70	-.01008	-.07842	-.2568	-.5891	-1.111	-7.241	-19.28	-35.61	-54.17	-73.45	-92.89	-110.9	-128.3	-144.8	-160.2	-174.7
80	$-.0^{2}7420$	-.05803	-.1911	-.4415	-.8388	-5.725	-16.06	-31.19	-49.62	-69.91	-90.93	-111.9	-132.5	-152.2	-171.2	-189.2
90	$-.0^{2}5577$	-.04378	-.1448	-.3360	-.6418	-4.517	-13.14	-26.47	-43.60	-63.37	-84.71	-106.8	-129.0	-150.8	-172.2	-192.8
	VALUES OF h AT ABUTMENT															
10	24.98	53.99	74.15	89.04	100.6	133.7	149.6	159.0	165.2	169.5	172.8	175.3	177.3	179.0	180.4	181.5
20	4.935	24.85	51.42	77.14	99.84	175.1	216.2	242.3	259.8	273.7	283.9	292.0	298.6	304.0	308.6	312.5
30	1.052	7.024	19.11	35.85	55.11	147.2	211.1	254.5	285.7	309.1	327.4	342.0	354.1	364.2	372.7	380.0
40	.3064	2.215	6.653	13.87	23.61	91.62	158.5	211.2	251.8	283.4	308.5	329.0	346.0	360.4	372.6	383.2
50	.1067	.7960	2.488	5.427	9.703	47.65	97.52	144.9	152.1	219.0	246.8	270.2	289.8	306.6	321.1	333.7
60	.04007	.3040	.9691	2.163	3.964	22.33	51.70	84.46	115.9	144.1	168.8	190.3	208.9	225.0	239.2	251.7
70	.01470	.1126	.3634	.8218	1.528	9.350	23.56	41.44	60.38	78.78	96.23	111.3	125.2	137.6	148.7	158.6
80	$.0^{2}4314$	.03330	.1083	.2470	.4637	2.993	7.993	14.85	22.72	30.88	38.89	46.47	53.51	59.99	65.91	71.31
90	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	VALUES OF m AT ABUTMENT															
10	.2421	.4950	.6450	.7367	.7935	.8559	.8057	.7388	.6752	.6185	.5692	.5264	.4892	.4567	.4280	.4026
20	.2045	1.001	2.013	2.938	3.702	5.737	6.348	6.443	6.291	6.123	5.885	5.638	5.395	5.163	4.943	4.736
30	.1058	.6931	1.833	3.406	5.139	12.58	16.63	18.61	19.49	19.76	19.69	19.42	19.04	18.60	18.13	17.64
40	.06094	.4341	1.285	2.640	4.432	16.07	26.07	32.74	36.87	39.34	40.72	41.37	41.55	41.41	41.05	40.54
50	.03854	.2842	.8777	1.893	3.345	15.54	30.17	42.64	42.68	58.73	63.38	66.54	68.59	69.82	70.46	70.66
60	.02594	.1949	.6152	1.359	2.468	13.26	29.36	45.96	60.51	72.34	81.58	88.63	93.92	97.80	100.6	102.5
70	.01820	.1383	.4424	.9920	1.830	10.75	26.04	44.12	62.00	78.10	92.23	103.3	112.5	119.9	125.8	130.4
80	.01311	.1004	.3242	.7341	1.368	8.519	21.98	39.49	58.47	77.05	94.11	109.2	122.2	133.2	142.5	150.1
90	$.0^{2}9594$	.07397	.2404	.5481	1.029	6.677	18.01	33.83	52.21	71.37	90.04	107.4	123.2	137.1	149.4	160.0
	VALUES OF v AT ABUTMENT															
10	-995.6	-990.5	-986.9	-984.3	-982.3	-976.4	-973.4	-972.0	-970.9	-970.1	-969.5	-969.1	-968.7	-968.4	-968.2	-968.0
20	-998.2	-990.9	-981.3	-971.9	-963.7	-936.3	-921.3	-911.8	-905.4	-900.4	-896.7	-893.7	-891.3	-889.3	-887.7	-886.2
30	-999.4	-995.9	-989.0	-979.3	-968.2	-915.0	-878.1	-853.0	-835.0	-821.5	-811.0	-802.5	-795.6	-789.8	-784.8	-780.6
40	-999.7	-998.1	-994.4	-988.4	-980.2	-923.1	-867.0	-822.8	-788.7	-762.2	-741.1	-723.9	-709.6	-697.6	-687.3	-678.5
50	-999.9	-999.1	-997.0	-993.5	-988.4	-943.2	-883.8	-827.3	-818.8	-739.0	-705.9	-678.0	-654.6	-634.6	-617.3	-602.3
60	-999.9	-999.5	-998.3	-996.3	-993.1	-961.3	-910.5	-853.7	-799.3	-750.4	-707.6	-670.5	-638.3	-610.2	-585.7	-564.0
70	-1000.	-999.7	-998.5	-997.7	-995.8	-974.3	-935.3	-886.1	-834.1	-783.6	-735.6	-694.2	-656.0	-622.0	-591.5	-564.3
80	-1000.	-999.8	-999.4	-998.6	-997.4	-983.0	-954.7	-915.8	-871.2	-824.9	-779.5	-736.5	-696.5	-659.8	-626.2	-595.6
90	-1000.	-999.9	-999.6	-999.1	-998.3	-988.8	-968.8	-939.7	-904.2	-865.3	-825.3	-785.8	-747.8	-712.0	-678.4	-647.2

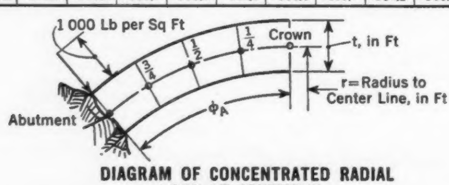


DIAGRAM OF CONCENTRATED RADIAL LOAD AT ABUTMENT

Thrust, Moment, Shear, in Lb, and Ft-Lb

H = h

M = m r

V = v

TABLE 9(*b*).—Radial Deflections for Concentrated Radial Load at Abutment

ϕ_A	VALUES OF t/r															
	.025	.050	.075	.100	.125	.250	.375	.500	.625	.750	.875	.1000	1.125	1.250	1.375	1.500
VALUES OF K_1 AT CROWN																
10	-808.3	-1434	-1592	-1653	-1633	-1726	-1736	-1740	-1742	-1744	-1745	-1745	-1746	-1746	-1746	-1747
20	1053	258.4	-344.8	-730.7	-974.1	-1412	-1518	-1561	-1583	-1596	-1605	-1611	-1616	-1619	-1622	-1624
30	1415	1125	708.4	442.5	143.2	-726.3	-1050	-1195	-1272	-1318	-1349	-1370	-1386	-1398	-1407	-1415
40	1501	1376	1217	1038	851.5	55.45	-410.9	-670.5	-823.2	-919.9	-985.1	-1031	-1066	-1092	-1113	-1129
50	1537	1469	1385	1288	1182	454.1	191.5	-112.6	-466.4	-459.8	-558.1	-631.2	-686.6	-729.7	-763.9	-791.7
60	1561	1517	1461	1407	1343	984.4	642.0	364.4	152.6	- 65.01	-126.7	-218.8	-290.8	-347.9	-394.1	-432.1
70	1581	1550	1514	1475	1433	1194	946.0	740.5	535.4	383.3	264.8	163.3	83.97	19.18	- 34.32	- 78.99
80	1601	1576	1550	1522	1449	1326	1170	978.4	826.2	695.5	584.8	491.7	414.7	349.5	294.5	247.7
90	1621	1674	1581	1559	1537	1417	1288	1161	1043	937.2	844.2	763.6	694.0	633.6	582.4	537.7
VALUES OF K_1 AT $\frac{1}{4}$ POINT																
10	-728.0	-1466	-1608	-1662	-1689	-1727	-1736	-1740	-1742	-1743	-1744	-1744	-1745	-1745	-1745	-1746
20	728.0	38.14	-490.2	-833.0	-1045	-1433	-1528	-1566	-1586	-1597	-1605	-1610	-1614	-1617	-1620	-1622
30	1039	796.3	459.6	210.8	- 48.64	-809.3	-1094	-1221	-1289	-1330	-1357	-1375	-1389	-1399	-1407	-1414
40	1108	1009	878.5	728.7	571.5	-109.9	-514.0	-739.0	-871.8	-955.8	-1012	-1053	-1082	-1104	-1122	-1136
50	1089	1083	1017	939.4	853.4	395.8	22.35	-236.6	-591.9	-531.6	-617.0	-679.2	-726.3	-762.8	-791.7	-815.1
60	1150	1118	1079	1034	984.6	696.2	414.1	183.4	66.01	-126.3	-226.7	-303.5	-363.6	-411.0	-449.3	-480.6
70	1162	1140	1112	1086	1052	868.7	671.1	490.7	338.6	214.8	120.3	35.75	- 28.63	- 81.10	-124.3	-160.3
80	1172	1156	1138	1118	1096	972.3	835.6	703.0	583.3	480.1	392.8	382.3	258.5	207.9	161.8	127.7
90	1183	1170	1156	1141	1125	1037	931.8	826.4	721.8	629.7	546.0	524.1	493.2	447.2	408.8	373.9
VALUES OF K_1 AT $\frac{1}{2}$ POINT																
10	-1192	-1556	-1650	-1687	-1705	-1731	-1737	-1739	-1740	-1741	-1741	-1741	-1742	-1742	-1742	-1742
20	-128.7	-547.1	-880.1	-1098	-1238	-1494	-1557	-1581	-1593	-1600	-1605	-1608	-1610	-1612	-1613	-1614
30	53.51	- 70.71	-256.6	-407.3	-563.5	-1035	-1215	-1296	-1338	-1363	-1379	-1390	-1397	-1403	-1408	-1411
40	83.89	46.78	- 13.45	- 88.94	-172.3	-557.6	-795.8	-929.4	-1010	-1057	-1089	-1111	-1128	-1141	-1149	-1156
50	88.32	77.05	55.04	24.32	- 67.01	-381.1	-436.3	-576.6	-841.5	-736.0	-781.4	-814.3	-838.9	-855.6	-869.6	-880.6
60	86.05	83.58	75.95	63.94	47.69	- 68.13	-197.0	-306.7	-391.6	-455.2	-502.5	-538.1	-565.2	-586.1	-602.5	-615.5
70	81.20	82.10	80.30	76.07	69.68	14.41	- 58.66	-130.2	-191.8	-241.9	-276.9	-312.9	-337.4	-356.7	-372.1	-384.4
80	75.10	77.21	77.89	77.23	75.36	68.01	14.71	- 25.83	- 35.79	- 96.82	-124.3	-208.5	-243.4	-271.1	-295.6	-319.0
90	74.77	70.43	71.92	72.50	72.16	58.03	25.82	- 11.79	- 54.59	- 96.56	- 93.49	- 74.73	- 24.60	- 52.55	- 58.13	- 62.74
VALUES OF K_1 AT $\frac{3}{4}$ POINT																
10	-1539	-1672	-1708	-1722	-1729	-1737	-1737	-1737	-1737	-1737	-1737	-1737	-1736	-1736	-1736	-1736
20	-1177	-1273	-1376	-1448	-1495	-1580	-1598	-1603	-1604	-1604	-1603	-1605	-1602	-1602	-1601	-1636
30	-1117	-1121	-1154	-1184	-1221	-1344	-1388	-1404	-1410	-1412	-1411	-1410	-1409	-1408	-1406	-1405
40	-1121	-1102	-1093	-1092	-1097	-1146	-1181	-1197	-1202	-1202	-1201	-1197	-1194	-1191	-1187	-1184
50	-1131	-1109	-1091	-1078	-1068	-1048	-1043	-1039	-1160	-845.0	-1015	-1006	-997.0	-988.9	-981.4	-974.5
60	-1141	-1121	-1103	-1087	-1073	-1019	-981.2	-953.5	-927.8	-904.7	-883.9	-865.1	-848.2	-832.9	-819.1	-806.5
70	-1153	-1135	-1119	-1103	-1089	-1026	-979.2	-927.9	-886.7	-849.8	-813.2	-785.4	-760.6	-736.8	-715.4	-696.0
80	-1162	-1147	-1133	-1119	-1106	-1044	-988.3	-917.3	-886.7	-841.2	-799.4	-761.4	-726.8	-695.4	-667.0	-641.1
90	-1174	-1161	-1149	-1137	-1126	-1080	-1045	-1019	-999.2	-982.7	-968.0	-830.1	-737.9	-702.0	-668.3	-637.7
VALUES OF K_1 AT ABUTMENT																
10	-1777	-1768	-1762	-1757	-1753	-1743	-1738	-1735	-1733	-1732	-1731	-1730	-1729	-1729	-1728	-1728
20	-1782	-1769	-1752	-1735	-1720	-1671	-1645	-1628	-1616	-1607	-1600	-1595	-1591	-1587	-1584	-1582
30	-1784	-1778	-1765	-1748	-1728	-1633	-1567	-1523	-1491	-1466	-1448	-1432	-1420	-1410	-1401	-1393
40	-1785	-1782	-1775	-1764	-1750	-1648	-1548	-1469	-1408	-1361	-1323	-1292	-1267	-1245	-1227	-1211
50	-1785	-1783	-1780	-1773	-1764	-1684	-1578	-1477	-1462	-1319	-1260	-1210	-1168	-1133	-1102	-1075
60	-1785	-1784	-1782	-1778	-1773	-1716	-1625	-1524	-1427	-1339	-1263	-1358	-1139	-1089	-1045	-1007
70	-1785	-1784	-1783	-1781	-1778	-1739	-1669	-1648	-1489	-1399	-1313	-1239	-1171	-1110	-1056	-1007
80	-1785	-1785	-1784	-1782	-1780	-1755	-1704	-1635	-1555	-1472	-1391	-1315	-1243	-1178	-1118	-1063
90	-1785	-1785	-1785	-1784	-1784	-1777	-1762	-1738	-1707	-1672	-1634	1460	-1335	-1271	-1211	-1155

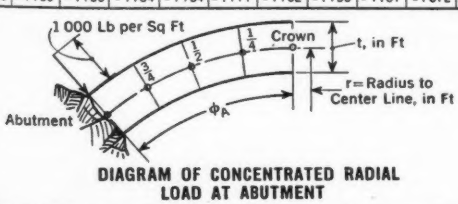


DIAGRAM OF CONCENTRATED RADIAL LOAD AT ABUTMENT

Radial Deflection, in Ft

$$\Delta_r = \frac{K_1}{E_c}$$

E_c = Modulus of Elasticity of Concrete in Direct Stress, Lb per Sq Ft

General Assumptions Made in Analyzing Arch Elements

In their paper in this Symposium, Messrs. Houk and Keener present a thoroughly inclusive list of assumptions that affect the design of masonry dams. Those that apply to circular arch dams of uniform depth are as follows:

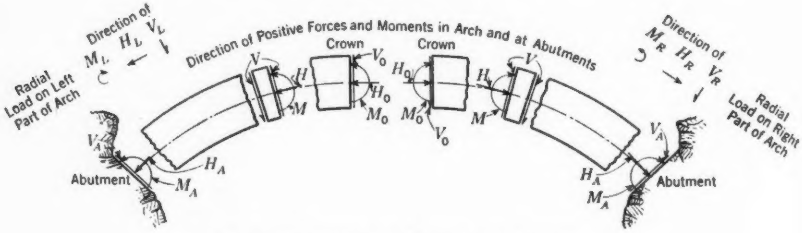


Fig. 2.—Direction of Forces and Moments

(*a*) The arch material is homogeneous and isotropic;

(*b*) Hooke's law applies, and the proportional elastic limit is not exceeded;

(*c*) A plane section before bending remains plane after bending;

(*d*) Direct stresses have a linear variation from extrados to intrados;

(*e*) The modulus of elasticity in direct stress is the same for tension and compression;

(*f*) Temperature strains are proportional to temperature changes;

(*g*) The temperature change occurs uniformly throughout an arch;

(*h*) The abutments of arch elements are radial planes;

(*i*) The ratio of the modulus of elasticity in direct stress (E) to the shearing modulus of elasticity (G) is expressed by

$$\frac{E}{G} = 2\,(1 + \mu) \qquad (2)$$

in which μ = Poisson's ratio;

(*j*) The symbol K expresses the ratio of the detrusion caused by the actual shear distribution to that caused by an equivalent shear distributed uniformly. Assuming that $K = 1.25$ and $\mu = 0.2$, the ratio $\frac{K}{G} = \frac{1.25 \times 2.4}{E} = \frac{3}{E}$, which is the ratio included in Tables 3 to 9 (Appendix).

Further Assumptions Applying to the Tables.—In Tables 3 to 9, furthermore, it is assumed that:

(*k*) The arch is circular and of uniform thickness (that is, t and r are held constant), and 1 ft wide;

(*l*) The arch and its loads are symmetrical about the crown section;

(*m*) The moment of inertia of a radial arch cross section, about a vertical line through its center of gravity is,

$$I = \frac{t^3}{12} \qquad (3)$$

Notation and Units

The notation and units, and the arguments, required to enter Tables 3 to 9 are given on each table. All dimensions are in units of feet; and forces and moments are in units of pounds and pound-feet. The process described in this paper is for a modified radial adjustment. For the amplified method the reader is referred to "Trial Load Method of Analyzing Arch Dams."[5]

PREPARATION OF FOUNDATIONS

By Charles H. Paul,[7] and Joseph Jacobs,[8] Members, Am. Soc. C. E.

Synopsis

It is assumed that only rock foundations are considered in this paper, except as indicated in the section headed "Special Conditions." The term "foundation" is taken to mean not only the entire area on which the dam rests, but also the areas upstream and downstream and out into the abutments adjacent to the limits of the superimposed structure, as far as the loading, seepage, or scour criteria may require.

Let it be stated at the beginning that no amount of text or treatise material can take the place of sound, experienced judgment in passing on conditions and requirements for dam foundations. All that the writers can hope to accomplish is to emphasize the more important requirements and to discuss, briefly, some general methods of treatment. Every foundation for an important dam is a problem in itself and should be given attention by men of broad experience and sound judgment in such matters.

Main Considerations

Assuming that preliminary investigations have indicated that a suitable foundation is available, the main considerations for the preparation of such foundation are: (1) Bearing capacity, (2) bond (with masonry), (3) sliding or frictional resistance, (4) control or reduction of uplift pressure, (5) prevention or control of seepage, and (6) prevention or control of scour from overflow or outlets. The general nature of these considerations is defined in this section, with methods of treatment presented in somewhat greater detail in subsequent sections.

Bearing Capacity.—For want of a better term, "bearing capacity" is used to indicate the supporting characteristics of the rock over large areas. In that respect it is synonymous with compressive strength; but the latter term usually applies to the strength of rock fragments or blocks, and there are numerous instances of hard rock so interspersed with seams that the blocks may readjust themselves under heavy loading and thus result in settlement or movement that would not be permissible in a dam foundation. Basaltic rocks are often in this condition, and decay in badly weathered rock sometimes follows horizontal and vertical seams, leaving blocks of sound, hard rock between. The important requirements for bearing capacity are not only the compressive strength of the rock itself, but also its adequate bearing strength over the entire area of the foundation.

[7] Cons. Engr., Dayton, Ohio.
[8] Cons. Engr., Seattle, Wash.

In general, most firm, non-decomposed rocks have sufficient bearing capacity for dam foundations, but special consideration is necessary for high dams. Seamy, fissured, faulted, and stratified rocks present special problems which must be dealt with in accordance with the loading and other conditions. Some of the approved methods for treating such foundations are discussed subsequently under other headings. Cavernous rocks present so many special problems that they are dealt with (although necessarily in a brief manner) under the heading "Special Conditions." It is sometimes necessary, and practicable, by washing out and grouting seams, and by "dental work," to reinforce or solidify a foundation that is quite unsatisfactory in its natural state, so as to make it entirely adequate for the purpose desired. Such work may have to be extended over the entire area and into the subfoundation to considerable depth in order to satisfy the requirements fully. Obviously, care and good judgment are essential in its execution.

Bond.—It now is generally accepted as a fact that by the use of approved methods a tight bond can be secured at the contact between clean, hard rock and the masonry of a dam. To accomplish this, however, the rock surface must be absolutely clean. A grout wash or mortar coat, preferably spread with wire brushes, immediately preceding the placing of the masonry, gives added assurance of accomplishing the desired result. A thin film of dirt, oil, or grease, or other similarly objectionable foreign matter, may weaken or destroy what otherwise should result in a tight, firm bond capable of meeting all requirements for contact between the dam and its foundations.

Fragile, crumbly rocks present special problems, although their bearing capacity may be satisfactory. Shales also require special study and treatment as to bonding methods. In cases where the character of the foundation is such that a firm, tight bond cannot be assured, then keying into the foundation, and other suitable modifications of the design of the dam, may be sufficient to secure safe construction.

Sliding or Frictional Resistance.—On firm, hard rock, where a tight bond can be secured, there is no danger of sliding at the contact surface. In such cases, movement could occur only by shearing. However, in stratified rocks, where clay or soft seams of appreciable thickness occur below the prepared surface, the danger of sliding in the foundation itself must be taken into consideration. Cohesion and friction factors in such cases must be studied carefully. Washing out and grouting the seams may be practicable. Often it is necessary to set the dam deep into the foundation to secure a downstream shoulder that will serve as a buttress to prevent movement in that direction.

The precautions mentioned under the preceding subheading, "Bond," should also be given full consideration under this subheading in all cases where a tight bond cannot be secured between the dam and the foundation rock.

Control or Reduction of Uplift Pressures.—Grouting foundation seams, and the combination of a grout curtain and drainage, are the measures usually relied upon to control or reduce uplift pressure. The extent to which uplift pressure and its distribution should be provided for in the final design will depend on the character of the foundation rock, the probable success of the grouting operations, and the probable effectiveness of the drainage system.

Prevention or Control of Seepage.—Measures taken to control uplift pressures work automatically (as far as they go) to help prevent or control seepage. Seepage control, however, may require much more extensive work. Seepage may find its way not only under the dam, but also out around or through the abutments. Upstream earth blankets or concrete aprons, washing and grouting seams, tunneling out and plugging large seams, cutoffs or grout curtains in foundation and abutments and out beyond the limits of the structure, all have their place under certain conditions. Rock blankets on downstream banks, or other forms of downstream drainage, may be required to control such seepage as cannot assuredly be prevented.

Protection Against Scour.—Protection against scour is a subject of great importance and involves many considerations. The hydraulic-jump principle, in the design of protection features, is the basis of the more common methods of treatment. Where a deep tailwater is assured, very little may be required in the way of special foundation treatment; but in many cases a properly designed apron is necessary, preferably designed by use of the hydraulic-jump formula, supplemented by model tests. In other cases, as at Grand Coulee Dam, in Washington, a specially designed bucket is effective.

Even so-called solid rock usually has some seams into which water can penetrate. Water in such seams acts effectively as a wedge; and the pounding of falling water acts as a mighty impact force on these water wedges and tends to disrupt the rock unless it is properly protected. If the tailwater depth is not sufficient protection, then the need for a concrete apron is indicated. Sometimes, when conditions are not severe, a comparatively thin concrete apron will blanket the rock seams and thus serve as sufficient protection. All such aprons should be so designed as to safely resist possible uplift pressures.

Investigations

The importance of thorough preliminary surface and subsurface explorations is obvious. As an aid to sound conclusions, a competent engineering geologist should have continuous contact with all such explorations, working in cooperation with the engineers. This will make available an intelligent interpretation of the geologic history, and of the origin, structure, and character of the rock, as they apply to the engineering requirements.

These preliminary investigations, however, are only a first step in the examination and study of the foundation. The observations and studies should be continued after the rock has been exposed by removal of overburden, and should include detailed surface and subsurface examinations up to the time of its final preparation. Even in the best foundations there are likely to be weak spots, fractured areas, slips, or seams, which require special study after full exposure and after all pertinent information is available as obtained by stripping, core drilling, open shafts, drifts, and other means.

More detailed exploration of subsurface conditions should follow the preliminary investigations of topography, geology, and preliminary borings, at sites where these early studies indicate favorable conditions. These more detailed explorations usually take the form of test pits, trenches, abutment drifts, and core drilling with suitable types of drilling machines.

Exploratory Drilling.—Various types of core drills are in use—the diamond drill, the shot drill, and some of the newer devices with cutting points of special alloy steels for use, mainly, in sedimentary rocks. The diamond drill is best for use in the harder rocks and is probably a little faster than the shot drill. It may be noted, too, that with the latter the shot is sometimes lost in considerable amounts in large open seams or crevices in the rock. There is now available an excellent light-weight portable diamond drill suitable for exploration in rough country, or in galleries or other places where space is limited. These little rigs require no tripod or other mounting except direct clamping to the casing pipe in the hole to be drilled. Good equipment and good operating technique are essential for best results. The more usual practice is to drill vertical holes; but inclined holes are entirely practicable and are frequently used in foundation exploratory work.

Although inclined holes are somewhat more difficult to drive, and cost a little more per linear foot than vertical holes, they are effective, particularly where a zone of some width is to be explored. For this reason, they are sometimes more desirable and more economical than the greater number of vertical holes that would be required to develop the same information. In bad foundations requiring extra precaution, or where vertical or nearly vertical seams are anticipated, a high percentage of exploratory coverage may be obtained by a drilling pattern which combines vertical holes and inclined holes, alternately.

The required number and depth of drill holes will depend mainly upon the local rock structure and upon the magnitude and importance of the proposed dam. They should be adequate, however, to reveal, definitely, the character of the foundation rock. They can generally be driven, at moderate cost, to practically any depth, and at practically any angle that reasonably may be required. The core should be labeled and indexed methodically, for both immediate and subsequent critical examination. All nonrecovery sections of the core should have dummy fillers so marked as to indicate the probable reason for the nonrecovery. As an aid to correct core interpretation, laboratory tests of the rock, as to its compressive strength, modulus of elasticity, hardness, soundness, solubility, permeability, and absorptive properties, are desirable.

In general, crystalline rocks, such as granite, and thoroughly indurated sedimentary rocks, such as quartzite, yield the best cores. The softer sedimentary rocks, such as loosely bound sandstone and shales, yield the poorer cores. A poor core recovery may mean a poor rock throughout; it may mean the piercing of extensive crush zones and seams in otherwise good rock; and it may mean too small a drill hole, or poor equipment, or poor technique in operation. Cores of less than about 1-in. diameter should generally be avoided because of their greater breakage tendency and their greater susceptibility to failure under inexpert drill operation. Diamond-drill cores of a diameter from $1\frac{1}{4}$ in. to 2 in. are the more desirable, and shot-drill cores, for best results, should generally be from 3 in. to 5 in. in diameter.

In drilling operations, the ground-water table should always be located, if possible, its elevation carefully noted, and its fluctuation during progress of the work recorded and studied. Similarly, any water flows or seepage encountered in drilling should be noted as to position and volume, for such data serve

as partial indexes of rock tightness. The major facts to be observed, and to have in mind, in the process of core drilling, may be listed as follows: (*a*) Depth of overburden; (*b*) depth to ground-water table; (*c*) depths at which drilling water escapes; (*d*) character and classification of the rock; (*e*) condition and quality of the rock; (*f*) evidences of processes affecting the rock, such as crushing, weathering or other forms of decay, and solution effects; (*g*) nonrecovery of core and the reasons therefor; (*h*) porosity and permeability of the rock, and seepage conditions in the rock structure as a whole; (*i*) any required tests to show depths and rates of leakage into hole from ground water; and (*j*) any required tests to show rate of leakage out of hole under drilling pressure or higher testing pressure.

Core Drilling Interpretations.—The purpose of core drilling is to secure, as nearly as possible, undisturbed samples, to predetermined depths, of the foundation rock that is to support the dam. Usually such drilling, in the aggregate, involves considerable expense. Frequently the cores may be impaired by undue breakage, or by undue grinding into granular particles, due to the use of poor or unsuitable drilling equipment or poor technique in operating the equipment. Such cores may easily be misinterpreted. Too often the interpretation of drill cores is treated as if it were of only casual importance, whereas much depends upon correct interpretation. It is as important and necessary as the securement of the cores themselves, and is the only justification for the considerable expense involved.

It is very desirable to have the drilling supervised continuously by an experienced man with a knowledge of engineering geology and also of the technique of drilling and of core recovery. This provides for intelligent interpretation and control as the work proceeds, including modification of the drilling program where advisable. It also affords opportunity to determine promptly the reasons for nonrecovery of core when such instances occur and, if equipment or technique rather than rock structure is responsible therefor, to apply the necessary remedies.

The percentage of core recovery, of course, is something of an index of rock character but by no means an infallible index. A good core recovery is a favorable indication, but a relatively poor recovery, for reasons mentioned in a previous paragraph, is not necessarily an indication of bad rock. From all of the foregoing it is clearly evident that core drilling calls for expert interpretation, and it should be appreciated that such interpretation requires a background of thorough knowledge and understanding of the engineering geology of the dam site. Without correct interpretation of results, it is obvious that the potential value of core drilling work is largely lost. Furthermore, incorrect or incomplete interpretation may lead to incorrect treatment of the foundation and even to incorrect design of the dam.

Large-Size Shot Drills.—The large-size shot drill is a piece of equipment that has become almost indispensable in the examination of important foundations.[9] By its use, a good check may be had on the interpretations of the small drill cores. This drill leaves a clean hole, 36 in. to 40 in. in diameter according to the size of the drill bit selected, with the rock walls undisturbed

[9] *Proceedings*, Am. Soc. C. E., June, 1939, p. 953.

and clearly exposed in their natural condition. A true and reliable picture is given, not only of the rock in place, but also of the size, character, and inclination of seams. Pockets or caverns, if intersected, are sliced through cleanly without disturbance, and are thus made accessible for further examination. It has been demonstrated that a 36-in. or 40-in. hole can be drilled to a depth of 50 to 60 ft without great difficulty. It is practicable to go to depths of 75 to 100 ft if necessary or particularly desirable. An observer may descend into these holes, by ladder or by sling, and examine the walls at leisure, and the results of such examinations are not confused by suspicions of artificial disturbances, as is often the case when shafts are driven by ordinary rock excavation methods, even where only light blasting is permitted.

FIG. 3.—VIEW OF LARGE CORE HOLE, TENNESSEE VALLEY AUTHORITY

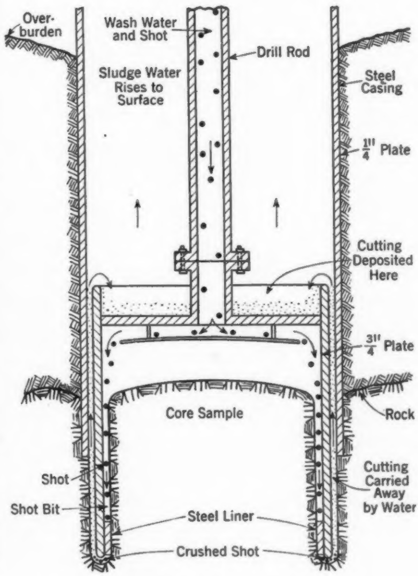


FIG. 4.—SKETCH SHOWING ACTION OF LARGE-DIAMETER CORE DRILL

Fig. 3 shows more clearly than can be expressed in words the advantage of these large holes as a means of subsurface investigation. It is of particular value in that it shows in one view the clean exposure of the rock, as well as the exposure of a cavernous solution channel and the convenient access to it.

Fig. 4 shows the method of operating this drill. The unit cost of drilling holes with this equipment varies considerably, depending upon the character of the rock, depth of hole, and the difficulty of removing the core; in rough figures it may be given as from $10 to $30 per ft. The character of the rock, of course, affects the speed of drilling, but another significant cost item is the removal of the core. One method of removing core is to drop a very small charge of powder down along one side of the core and crack it off in 2 to 3 ft lengths. Often the core can be cracked off by wedging. If the core cracks

through without fracturing otherwise, it is easily removed by means of a lifting device inserted into a small hole drilled down the core axis. In case the core breaks up into small pieces, the job of mucking out is slow, and becomes an important factor in the cost. In such cases it usually is economical to use the 40-in. drill, so that the workman is not so cramped for room in cleaning out the hole.

In ordinary hard rock, the core from these large-size holes may be removed in sections, properly matched and oriented, and laid out on the ground as an exhibit of the character and structure of the foundation rock; but by far the greatest value of this type of exploration lies in the opportunity to examine the rock in place exactly as Nature presents it for the use of Man.

The most common method of using these large drills is to set them up on the foundation rock, after the overburden has been removed (a dry-land process), thus penetrating rock only; but on some of the investigations of the Tennessee Valley Authority (TVA), the method has been adapted to operation from a barge for the purpose of exploring the submerged rock in river channels.[10] Such adaptability suggests a broadly widening field for the use of this large drill equipment. Its judicious use, on important jobs, is almost certain to yield valuable information.

Rough Excavation

The term "rough excavation" refers to the removal of all overburden and, furthermore, to the removal, by customary excavation methods, of those portions of the immediately underlying rock that are soft, weathered, badly fissured, or otherwise unsuitable for foundation purposes, this operation being extended fairly close to what is expected to be the final surface of the rock excavation. Rough excavation is to be distinguished from the scaling and trimming of the rock surface, which is the final step in the rock excavation process. The practical significance of this distinction lies mainly in the extent to which blasting is permissible in the final stages of the excavation operations.

It is important, and progressively more important with increasing height of the dam, that the dam structure be founded upon solid rock, entirely free from loose or shattered material. The reverse of this condition might easily result from careless or heavy blasting or any blasting at all in the immediate zone of final excavation. It is necessary, therefore (and customary), to limit the extent of permissible blasting and to define clearly such limitation in the specifications. A not unusual practice is progressive blasting limited to depths of about two thirds the total estimated remaining depth of excavation, with decreasing charges of explosives for the shallower depths, until within a foot or two of the foundation level, when blasting is prohibited and the remaining material is removed by barring, wedging, use of a paving breaker, or other means not requiring blasting.

An excellent example of a provision for control of blasting is that found in the specifications for construction of Grand Coulee Dam, which are as follows:

> "*Rock Excavation for Foundations for Dam and Power House.*—The excavation for the dam and power house shall be made to sufficient depth

[10] "Underwater Exploration with Calyx Drills," by Hendon R. Johnston, *Engineering News-Record*, March 24, 1938, p. 436.

to secure foundation on sound ledge rock, free from weathered material, open seams, or other objectionable defects, as determined by the contracting officer. All necessary precautions shall be taken to preserve the rock below and beyond the lines of excavation in the soundest possible condition. All blasting operations, the depth and size of holes, and the size and characteristics of the charge shall be subject to the approval of the contracting officer. The explosives shall be of such quantity and moderate power and used in such locations as will neither open up seams nor crack or damage the rock outside the prescribed limits of excavation. The firing of systems of blasts shall be controlled by the use of delay exploders. No hole for blasting shall be drilled more than two-thirds the depth of the proposed excavation. Where the depth of excavation is less than 15 feet, the holes shall not be larger than necessary to permit the passage of whole sticks of powder to the bottom of the hole and such holes shall not be sprung or chambered. As an excavation approaches its final lines the depth of holes for blasting and the amount of explosives used per hole shall be progressively reduced. Whenever, in the opinion of the contracting officer, further blasting is liable to injure the rock upon or against which concrete is to be placed, the use of explosives shall be discontinued and the excavation completed by wedging, barring, channeling, line drilling and broaching, or other suitable methods. The excavation for the base of the dam and power house at all elevations shall be so shaped and roughly stepped where necessary, as determined by the contracting officer, to produce the desired surface of contact between the concrete and the foundation rock."

A good practice in the excavation of dam foundations is to require that the overburden be removed sufficiently to permit an early inspection of the rock before its actual excavation is begun. This is desirable in order that a more intelligent program, and a more detailed procedure, as to rock excavation and surface treatment of the rock, may be outlined in advance. In some cases, it may be desirable to explore the rock further with jackhammer or with large-diameter drills. A program thus outlined may need to be revised, upon full exposure of the rock, but that possibility does not at all lessen the desirability of securing the advance information suggested herein. Such advance information is always valuable.

Depending upon the character of the rock, the extent of weathering that has occurred, the steepness of the abutments, etc., the relative amounts of rough excavation, and of scaling and trimming, on different jobs, are widely variable. On Boulder Dam, for example, these relative amounts were 68% and 32%, respectively, of the total rock removed, and similar data for the Owyhee Dam, in Oregon, were 85% and 15%, respectively. On the Arrowrock Dam, in Idaho, after removal of the overburden, the volume of rock excavation required in the river-channel section was negligible and only on the abutments was there need for appreciable quantities of rough rock excavation or of scaling. Although rough excavation almost always constitutes the major part of the total foundation excavation, it is apparent from the foregoing that scaling and trimming may become a significant item of cost in some cases.

Final Excavation and Finish

"Scaling and trimming" is the final removal of slabby or "drummy" rock, including all pieces partly loosened by the work of rough excavation or partly

separated from the main rock mass by seams or cracks. Should any weathered or partly decomposed rock remain after the rough excavation is finished, it also should be removed during the scaling and trimming operations. All this work should be done with great care so as not to leave on the foundation surface any rock other than that which is an integral part of the rock mass. It is customary, in this operation, to prohibit or strictly limit the use of powder, requiring the scaling and trimming to be done with picks, paving breakers, bars, and wedges.

Areas of low bearing capacity, steeply inclined seams, faults, or crush zones, in an otherwise good foundation, always call for special consideration. Their relative importance depends on their size or area; whether they are, or can be, fully confined; whether they can be protected against further deterioration; and whether their bearing capacity, even though below the average of the foundation, is still sufficient to carry the load without undue yielding. Usually these doubtful areas are cleaned out to sufficient depth and refilled or plugged with concrete, in which cases their size, shape, and condition control the depth to which excavation should go. It is often desirable that this cleaning out and plugging be followed by grouting as an additional precaution, particularly near the upstream face of the dam where seepage control is important.

It is often assumed that relatively small areas, whether treated or not, that might not carry their share of the load will be spanned by the masonry of the dam and thus be rendered harmless. Often this may be true, but any such condition means some concentration of stresses in the masonry which should not be overlooked. It may be negligible, or it may be important, depending upon the extent and character of the unbalanced distribution of loading. In general, it may be stated that, in an otherwise good foundation, areas of low bearing capacity are troublesome, but if given proper treatment they usually are not serious.

The prepared foundation surface should be free from sharp angles of such extent as might tend to induce shrinkage or shearing stresses in the concrete of the dam. Top edges of benches, or the like, should be chamfered approximately 45° so as not to be objectionable in this respect. Pinnacles or sharp projections should be knocked off, and prominent knobs should be flattened off for the same reason. Deep potholes and narrow gorges should be filled with concrete, so that it can set and cool and take its initial shrinkage before the concrete of the dam is placed. Any abrupt changes in elevation of the foundation tend to induce shearing stresses in the concrete of the dam because of unequal shrinkage. Corrective measures may be applied in preparing the foundation and in placing the concrete in short lifts against the rock face so as to permit initial shrinkage to take place in the lower lifts before the higher ones are placed. Fig. 5 shows a part of the foundation of Grand Coulee Dam, scaled and trimmed and ready for final cleaning.

Following the scaling and trimming, the foundation surface should be thoroughly cleaned of all loose particles, including the very finest of the chips, sand, and dirt. Immediately in advance of placing concrete, the foundation should be scrupulously clean. To accomplish this it is a common practice to use a strong air and water jet, powerful enough to dislodge any particles loosely

attached to the rock surface. The dirty water that accumulates in puddles is removed in buckets or by other convenient methods. The washing and scrubbing process should be continued until the water in these puddles is clear, and entirely free from dirt. In the final cleaning process, sponges are commonly

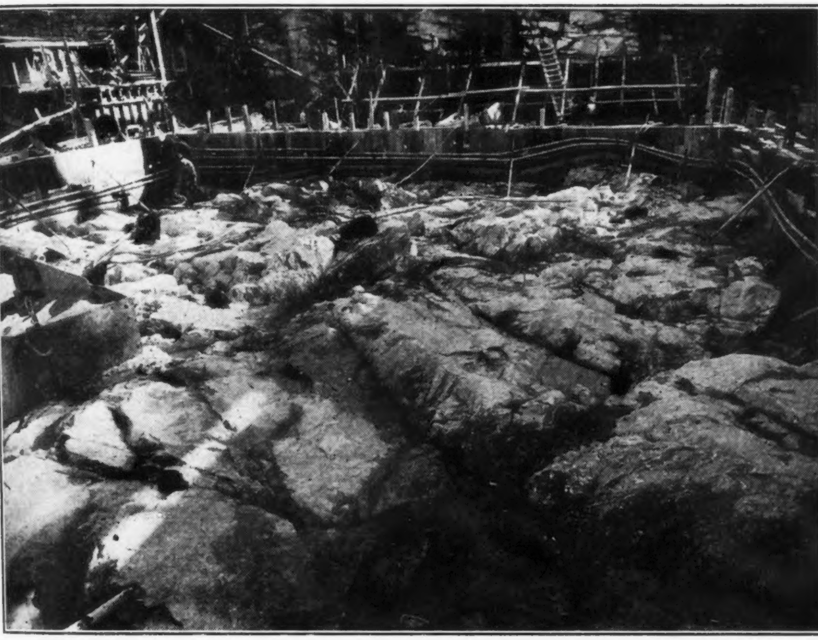


FIG. 5.—VIEW OF THE FOUNDATION OF GRAND COULEE DAM, READY FOR FINAL CLEANING

used to sop up the last of the wash water. Too much emphasis cannot be given to the importance of having an absolutely clean foundation on which to place the concrete of the dam. The slightest film of dirt, oil, or grease may prevent a tight bond, and to a large extent defeat the purpose of the vigilant supervision and the expensive work involved in foundation preparation.

Where well-defined cutoffs are required, or in other places where it is necessary to adhere strictly to prescribed lines, line drilling (or channeling and broaching in shale or the softer rocks) is often resorted to. This is expensive work and should be required only where adherence to prescribed lines is essential. Frequently, in excavation for cutoffs, for example, it may be practicable to allow some tolerance for overbreak. On the other hand, in seamy rocks where the effect of blasting is likely to extend beyond the prescribed lines, these more expensive methods are justified on the basis of additional safety and economy. The faces of downstream shoulders, where required to furnish resistance against sliding, are often best prepared by means of line drilling.

Grouting and Drainage

This paper would be strikingly incomplete without some reference to grouting of foundations. Therefore, although James B. Hays, M. Am. Soc. C. E., has presented a complete paper that deals with the general subject,[10a] there is given, in this section, a brief discussion of the writers' concept of the more essential features of cement grouting as applied to dam foundation preparation. No comments are made on asphalt grouting, clay grouting, chemical grouting, or other forms, for the reason that they are not commonly used in treating foundations for masonry dams.

Although of relatively modern application, cement grouting is now a practically universal procedure in masonry-dam foundation work, its purpose being to consolidate the underlying rock, and to reduce seepage and uplift pressure. The exploratory drilling work, and its correct interpretation, discussed in preceding sections, are indispensable precedents to the planning of any intelligent program for the grouting and drainage of a dam foundation. The large-size drill hole also provides invaluable information in this connection. It is also desirable that some preliminary studies be made to determine the range of grouting pressures to which different sections of the foundation rock may be subjected safely. Grouting pressures, the consistency of the grout, and the speed of injection are all important elements of foundation grouting.

Pressure Control.—Grouting pressures must be controlled carefully to insure against possible displacement of the rock. They must be adapted to the depth of hole, distance from exposed rock faces, and the character of the rock with respect, particularly, to open joints, lift seams, crush zones, etc.[e] Generally, in considering the subject, and in the preparation of specifications, a distinction is observed between low-pressure grouting and high-pressure grouting. The former, with pressures ranging, in good rock, from about 50 lb per sq in. to perhaps 150 lb per sq in., is applicable to shallow hole grouting in general—say less than 40 ft in depth, and for the consolidation of upper strata rock to prevent the escape of grout from the deeper, high-pressure grouting to be done later. In the latter, pressures range generally from 200 lb per sq in. to 600 lb per sq in. There is danger of rock dislodgment with these extremely high pressures, and the utmost caution must be exercised in their application. Frequent observations, by means of precise levels, the tiltmeter, or other device, to detect displacement, should be made during the process of grouting wherever there is danger of rock movements. Although the proper depth of hole depends mainly upon the character of the rock structure, the deep holes usually have a length about one fourth the hydrostatic head at maximum water level behind the dam. Deeper holes are sometimes necessary.

Washing Out Seams.—Before grouting is begun, the grout hole should be cleaned with compressed air and water and at the same time effort should be made to clean the seams in the rock whenever that is practicable. For narrow seams filled with gouge or stiff clay it is neither practicable nor necessary; but for the wider seams, filled with silt or with products of solution, it often is practicable and should be done. The process involves the application of compressed air and water, as already mentioned, a carefully devised pattern of

[10a] "Improving Foundation Rock for Dams," by James B. Hays, *Civil Engineering*, May, 1939, p. 309.

hole arrangement, and a hookup that permits a rapid reversal of flow between holes and through the seams. A good example of the successful application of this procedure is in the foundation treatment of Norris Dam.[11] The limiting pressures to be used in washing out grout holes and seams are practically the same as those previously defined for the grouting.

Grout Curtain.—Usually, although not always, a grout curtain parallel to, and near the upstream face, of the dam, effected by a single or a multiple line of grout holes, will suffice for such foundation consolidation as may be required for seepage reduction purposes. The depth of holes has already been discussed. The spacing of holes may vary from $2\frac{1}{2}$ ft to 5 ft or more, depending upon the character of the rock and the depth of reservoir water behind the dam. Where more than a single line of grout holes is used, the holes should be staggered with those of the adjacent line. For high dams two or three grout lines, with shallow, intermediate, and deep holes, are the usual practice. The shallow holes are grouted first, then the intermediate, and finally the deep holes, the latter generally being deferred until a considerable height of concrete has been placed. Good evidence as to the effectiveness of grouting operations may usually be obtained by first grouting holes with a wide spacing, say 20 to 40 ft, and then using the intermediate holes as test holes by comparing their acceptance of grout with those first grouted. For a single line of deep holes, stage grouting is generally desirable and is usually specified.

Stage Grouting.—When a single line of deep or moderately deep holes is used for developing a grout curtain, and in other places when such holes are to be drilled (particularly where the seamy character of the rock demands special care in grouting), a process of stage or successive drilling and grouting is often adopted. This process consists of the following successive operations: Drilling the hole to a depth of from 10 ft to 40 ft, depending upon the character of the surface rock; grouting at that depth under relatively low pressures; cleaning the grout hole and drilling it to farther depth; grouting to the new depth under somewhat higher pressure, and so on until the ultimate depth of hole is reached; and finally grouting it under high pressure. The purpose and the advantage of stage drilling and grouting are that they consolidate the successive rock zones from the surface downward, under increasing grout pressures, thus making the final grouting under high pressure effective by insuring against undue escape of grout through the upper rock zones.

Special Grouting.—Special grouting is often required on steep abutments; where major surface seams, crevices, or fractures in the rock are in evidence; where consolidation of blocky rock is required; and, in fact, at any questionable sections of the foundation, wherever they may occur. On the abutments, however, where, in the absence of grouting or other consolidation or cutoff provisions, the percolation path may be relatively short, a program of special grouting is usually desirable. This condition, at minimum, involves shallow grouting back of the immediate faces of the abutments, but, if the rock is badly seamed and jointed, it may also involve grouting operations outside of, and beyond, the boundaries of the abutment contact areas. In some instances, too, it may be desirable to drive a tunnel into the abutment from which to accomplish the

[11] *Engineering News-Record*, November 21, 1935; also *Proceedings*, Am. Soc. C. E., March, 1940, p. 385.

grouting and drainage required. Undue seepage, through and around the abutments, should be prevented, and the special grouting mentioned herein, together with properly placed concrete cutoff walls, is an effective means to that end.

Drainage.—There are few, if any, high masonry dams that do not require some provision for drainage, and such provision is now the practically universal practice in dam construction. Its purposes are to relieve upward pressure, thereby increasing the stability of the dam, and to pick up and safely conduct to the river channel below the downstream toe of the dam any seepage water that may find its way through or beneath the dam. It is accomplished, usually, by means of a series of drainage holes, 3 in. to 6 in. in diameter, about 10 ft on centers, and drilled into the foundation rock to depths of from 25 ft to 50 ft or more, depending upon the character of the rock structure and the depth of the adjacent grout holes. The holes are located on a line parallel to, and a short distance downstream from, the grout curtain. One or more additional lines of drainage holes may be required in some instances. The seepage water that wells up through the drainage holes, usually into a drainage gallery in the dam, is delivered, through transverse galleries and lesser conduits, to the downstream face of the dam close to the surface of tailwater. In placing the base concrete of the dam, metal pipes may be installed, or holes may be formed otherwise, to facilitate the subsequent drilling of the drainage holes, and these holes should not be drilled until all grout holes, which might possibly be connected by seams to the drainage holes, have been drilled and grouted.

Special Abutment Treatment

In general, the problem of preparing abutments is based on the same requirements as that of preparing the remainder of the foundation. As a matter of fact, the dividing line between the foundation and the abutment is frequently an imaginary or arbitrary one, and depends largely on how the abutment slope flattens out to meet the generally horizontal position of the bottom area. In a broad sense, the entire area upon which the dam rests, and the adjacent areas which are directly affected by the proposed structure, should be considered as foundation. In this section, however, the term "abutment" refers only to the steeper foundation slopes on either side of the bottom area.

Horizontal, or nearly horizontal, seams or solution channels, if any, are usually exposed by the abutment excavation and therefore are accessible for methods of direct treatment. Where such seams are of sufficient importance, they may be tunneled out and filled with concrete, or, where control of seepage is the main consideration, several cutoff drifts properly spaced upstream and downstream may be effective. In such cases a drift at the upstream face of the dam should be driven as far as may be necessary to force a percolation path of sufficient length to prevent undue seepage, or to a point where extension by grouting might be more practicable. In some cases, depending upon the character of the seam, this one cutoff may be sufficient; in other cases it may be desirable to supplement it by others so spaced as to extend further the path of any seepage that might find its way past the upstream cutoff. Sealing off the faces of these seams by substantial concrete plugs is usually desirable, and often

this makes it possible to do effective grouting behind the seal. In seamy rocks this control of seepage, so as to force it out and around the main structure of the dam and well into the abutment, is one of the important factors in abutment treatment. Frequently the work required to accomplish this must extend well out beyond the limits of the structure.

Abutment excavation necessarily involves steep slopes or stepped benches. Steep slopes, although unavoidable, are undesirable because they cause unequal shrinkage within short distances in the concrete of the dam. Stepped benches

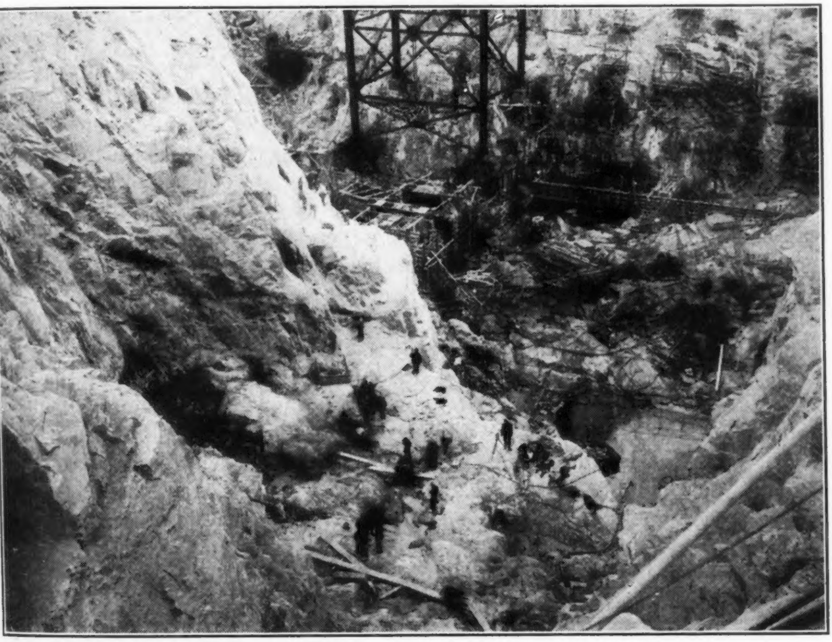


FIG. 6.—VIEW OF THE ABUTMENT, GRAND COULEE DAM, DURING THE PROCESS OF SCALING AND TRIMMING

are undesirable for the same reason, and require also that their edges be chamfered off to avoid a tendency to originate cracks. These are unfavorable conditions that occur to some extent in every dam abutment, and must be considered both in the preparation of the abutment foundation and in the design and construction of the dam itself. If the undesirability of steep slopes and benches is kept clearly in mind, together with the reasons why they are objectionable, then the measures adopted to meet the situation are most likely to be effective. These measures usually are limited to a combination of adjusting slopes, chamfering edges of benches, locating contraction joints so as to best fit the situation, and so placing the concrete that the effect of unequal shrinkage, or the tendency to crack from other causes, will be most effectively counteracted. Fig. 6 shows a part of one of the abutments of Grand Coulee Dam during the process of scaling and trimming.

Special Conditions

Sand and Gravel Foundations for Low Dams.—Consideration of building low masonry dams on sand and gravel foundations opens up a special field in dam design and construction. As far as preparation of foundation is concerned, the main considerations (see itemized considerations at the beginning of the paper) are the same as those for higher dams on rock foundations, but the very nature of these sand and gravel foundations calls for special study of these considerations as they apply to foundations of this type. It is the purpose herein to discuss only briefly each of these considerations and its significance as related to sand and gravel foundations.

As to bearing capacity (item (1)), a well-graded and compacted sand and gravel foundation may be sufficient for very low dams, and for dams of moderate height which follow the slab and buttress type of design or some similar special design which reduces unit stress by spreading the load on the foundation. Otherwise, in many cases, it is necessary to resort to the use of piles, and in such cases this pile foundation must be designed to carry the full load. Because of the necessary sequence in the process of construction, there is no practicable way of distributing the load between the sand and gravel and the piles, except to the extent that this is accomplished by friction between the piles and the material through which they are driven. The piles must take the entire superimposed load unless they settle sufficiently to transfer a portion of the load directly to the surface of surrounding material, but obviously, even slight settlement of the dam introduces other serious problems and should be avoided. In designing the pile foundation, a limitation that must be recognized, when the loading is heavy, is the minimum effective spacing of piles. Usually this should be not less than about 3 ft. To help secure a close contact between the foundation and the concrete of the dam, and in order to effect all possible presettlement of the foundation, it is desirable to have the foundation dewatered well below the prepared surface during the placing of the first few layers of concrete around and above the piles.

Of course, there can be no effective bond (item (2)) between sand and gravel foundations and the masonry of the dam. The design must take this into consideration. In some cases, not to effect a substantial bond but to obstruct flow of seepage water at foundation contact, it may be desirable to apply low-pressure grouting at the foundation contact after the base concrete has been placed in sufficient thickness to resist any uplift incidental to the grouting. To prevent the escape of the grout around the edges, resort may be had to "dry packing" or "stock ramming."

Resistance to sliding (item (3)) must be considered carefully. Where piling is not used, sufficient resistance may be developed by broadening the base, by setting the dam well into the foundation, or by substantial cutoffs properly designed, and properly located, so as to be effective in this respect.

Seepage (item (5)) is usually controlled by sheet-pile cutoffs along the upstream face of the dam. The piling preferably should be driven to rock or to a layer of impervious material. If this is not practicable, then the sheet-pile cutoff should be of sufficient depth to insure that the shortest possible seepage path will be long enough to prevent movement of the finer materials in the

foundation. Substantial, impervious, upstream blankets are effective aids in the control of seepage, in that they lengthen the path of seepage travel. These upstream blankets, usually of clayey earth, compacted by rolling or other effective means, are relatively inexpensive and when properly designed and constructed are very reliable. Concrete aprons are sometimes used for the same purpose. Special attention must be given, in the design, to the junction between these blankets or aprons and the masonry of the dam, so that a tight contact can be made there which will not be destroyed by slight settlement, and will not permit incipient leaks or piping.

In foundations of this type, a common measure of uplift pressure (item (4)) at any point is that obtained from an hydraulic grade line starting at the elevation of full reservoir at the upstream edge of the upstream blanket, or at the upstream face of the dam if there is no blanket, ending at the elevation of tailwater at the downstream limit of the structure, and adjusted between those points for such effects as may be conservatively assumed to result from sheet-pile or other cutoffs. In special cases—say, beneath the downstream third of the structure—downstream drainage may be desirable in order to insure that there will be no backing up of pressures, within the drained area, above the limits indicated by the theoretical hydraulic grade line.

Where a river-channel spillway section is a part of the design of a dam on sand and gravel foundations, as is often the case, protection against scour (item (6)) downstream from the spillway is of utmost importance. It is best controlled by a well-designed stilling pool, so developed as to be effective within the complete range of headwater and tailwater elevations. The downstream lips of these stilling pools are sometimes protected from undercutting by rows of round piling closely spaced, or by rock or concrete aprons; but a more effective method of protection is by a lip so designed as to form a ground roller that tends to pull back the loose channel material toward the edge of the lip—the exact opposite of undercutting. Lip designs to accomplish this result are quite dependable; in working them out, model tests are of great value.

Cavernous Rocks.—Cavernous rocks present many special problems which must be considered in connection with their geological structure; loading conditions to be met; extent and character of solution channels and their connections; and permissible loss of water from the reservoir, from the viewpoints of its value, of actual danger, or of the psychological effect of the seepage around or below the dam. In such rocks seepage may or may not endanger the structure, but even if it is not dangerous, such seepage, uncontrolled, may be extremely undesirable. By careful (and usually quite expensive) work, after adequate exploration and study, even cavernous foundations may often be treated so as to be acceptable for dams of moderate or intermediate heights; but it is obvious that such foundations are to be avoided if possible, and that they add a large, as well as an uncertain, element to the cost of the structure. Limestones, and basalts or other volcanic formations, offer the greatest possibility of cavernous structure.

Close drilling to sufficient depth usually is required to locate extensive caverns or solution channels. They can then be reached by large-diameter drills, or otherwise, cleaned out as may be necessary, and plugged with concrete

or some effective substitute. Spacing of drill holes for a grout curtain in cavernous rock may need to be as close as 1 or 2 ft, in order not to miss a space that might defeat the object of the curtain.

Shales and Other Stratified Rocks.—Shales and other stratified rocks require special study because of their diverse characteristics, and because of the liability of finding soft or permeable seams between the harder layers. A careful subsurface exploration is necessary to detect such weaknesses. The harder shales and most other stratified rocks are usually sufficient as to bearing capacity except for high dams.

Bonding to shales is best accomplished by a series of cutoffs to break up the continuity of the stratification, and by leaving the final trimming until just before covering with concrete, as most shales are likely to disintegrate quickly upon exposure. Other sedimentary rocks may or may not require special treatment for securing a tight bond, depending upon their characteristics.

Protection against sliding must be considered carefully in dealing with such rocks. Often the best plan is to set the dam deep into its foundation, so that downstream shoulders will hold it against movement in that direction. In such cases it is very essential that these supporting shoulders be protected against scour or disintegration. This may be accomplished by covering with concrete or other paving to protect against scour, or by covering with earth, a weather-proof coating, or some other similarly effective method to guard against disintegration.

Seepage may be controlled by cutoffs and, in some instances, by grouting, as already explained for other classes of rock. Whether grouting will be effective depends upon the character of the seams. In these types of rock, control of grout pressures is of special importance. Usually, by close spacing of holes, a dependable grout curtain can be secured, even if low pressures must be used and although few of the holes may take any appreciable quantity of grout.

Because of the position of seams and bedding planes in sedimentary rocks, uplift pressure must be given special consideration. Measures adopted for control of seepage are helpful in this respect, but horizontal seams, which may be continuous under the entire foundation, present an element of weakness that should not be ignored. Drainage holes, properly placed in such foundations, often give assurance of adequate control within limits anticipated in the design.

Stratified rocks are likely to be weak against attack by scour. Stilling pools and aprons should be designed with this in mind. Shales should be protected from exposure far enough beyond structure limits to insure against the effect of progressive disintegration back toward the dam.

Conclusion

It is the hope of the writers that this paper will succeed in centering the readers' attention on the basic considerations involved in the examination and preparation of foundations for masonry dams, as outlined herein. It is hoped also that sufficient emphasis has been given to the more usual conditions and methods of treatment to indicate the importance of a careful independent study of each foundation problem, to the end that no significant condition, and no necessary precaution, will be overlooked.

GEOLOGICAL PROBLEMS OF DAMS

By Irving B. Crosby,[12] Affiliate Am. Soc. C. E.

Synopsis

The geological and foundation problems involved in the economical construction of safe dams, and the possibilities of geological investigation, are discussed in this paper. The different stages of a complete geological investigation and the objectives of each stage are also discussed. Every branch of geology may be involved in investigation of a dam site and in addition many allied techniques and methods may be necessary. The paper is confined to the consideration of geological and foundation problems of masonry dams founded upon rock.

Foundation problems vary for different types of foundation rock and for different types of geologic structure. These problems are illustrated by examples of successful dams and dam failures on different types of rock and geologic structure. The purpose of these descriptions of dam failures is to point out the various geologic conditions which may cause failure and not to renew discussion of the conditions pertaining to each individual failure. The thesis of this paper is that the essential foundation conditions can be determined in advance, that at most sites safe dams can be built, and that dam failures are not unavoidable.

Introduction

Every dam rests upon geological formations and geological structures which, before the dam is built, are stable, or relatively so, but which, when the dam is finished and the reservoir is filled, are subjected to new and different stresses and conditions. The engineer must so design his dam in relation to these formations that they will continue to be stable under all possible conditions.

The problem derives much of its complexity from the fact that geological formations and structures are very seldom uniform for any considerable extent. They change vertically and horizontally, sometimes with great rapidity, sometimes gradually. The true conditions at a dam site must be determined and the design of the dam must conform to the conditions imposed by Nature. If these conditions are not met, the dam may fail. It may slide on a slippery bedding seam just below the foundations, as happened at Austin, Pa.; or the raised ground-water level may cause softening of the rock, as occurred at the St. Francis Dam, in California; or the water may pass under the dam through fissures and caverns, as at Hales Bar Dam on the Tennessee River, in Tennessee, or any one of a thousand possibilities may occur to destroy the dam or reduce its value. Not only is it necessary to determine the existing conditions

[12] Cons. Eng. Geologist, Boston, Mass.

of the rocks, but the possible effects of new stresses, new ground-water conditions, and new exposures to weathering and oxidation must be foreseen. Formations which are now stable may become unstable under new ground-water conditions or when exposed to weathering.

For the satisfactory interpretation of foundation conditions, all the resources of geological knowledge are necessary. Since it is seldom feasible to obtain all the desirable information about subsurface conditions, it is necessary to use geological principles and broad experience based upon work with every conceivable type of geological formation and geological structure in order to estimate conditions as accurately as possible. In reaching conclusions about subsurface conditions, the engineering geologist must make certain that no possible variation from his estimate can be such as to endanger the project. He must weigh the probability of accuracy of his statements, point out in what way actual facts may vary from them and what the effect of such variation would be, and must show what the worst possible conditions are. The practice of engineering geology, therefore, requires an unusually broad scope of geological training and knowledge of the constantly developing related sciences and techniques. Above all, it requires the ability to make important decisions on insufficient data with the assurance that those decisions are the best possible. Furthermore, in no type of geological work are the geologist's conclusions checked as remorselessly as when the excavations for a dam expose to view the underground conditions he has forecast, and when the filling of the reservoir puts a final test to his conclusions.

The use of geology in dam site investigation is now almost universal, but there is often failure to recognize the extreme importance of experience in this type of work. It is too often assumed that any geologist can give satisfactory results or that the geologist can be trained on the job and that prior experience is not necessary. A geologist trained on the job may make valuable observations, but, when any unusual condition arises, there is grave danger of serious mistakes.

To make a correct interpretation of subsurface conditions with whatever data are ascertainable, the engineering geologist must use methods of reasoning and of work different from those to which the engineer is most accustomed, for the precise and mathematical methods of engineering reasoning can seldom be used by the geologist. It is true that recent developments in geological and related sciences have given the geologist tools of quantitative measurement which partake of the methods with which the engineer is familiar; but the geologist must use these tools with great care since, if used without sufficient experience and knowledge, they may be dangerous. Both geologist and engineer, when questions relating to subsurface conditions are concerned, must be on their guard against a facile acceptance of seemingly precise quantitative results, for conclusions based on them can be no more accurate than the interpretation of subsurface conditions to which they are applied.

The duty of the geologist in investigations of dam sites should be to discover and make clear to the engineers in charge, as accurately and completely as possible, all the geological conditions and factors which may have any relation to the dam and to the construction work. When it is impracticable to

determine all the details of the geological conditions he must point out the worst possible conditions for which the engineer must design to insure safety of the dam. It should not be the duty of the geologist to make engineering decisions based upon geological facts. When the geologist has presented to the engineer a full description of geological conditions, the engineer can meet those conditions or decide whether it is not economical to meet them.

The geological investigation of dam sites naturally divides itself into three parts with different objectives: First, a reconnaissance investigation during the preliminary stage; second, the detailed investigation in the design stage; and third, continuing geological observation during the construction stage.

The reconnaissance investigation should be confined to a brief surface study without borings and should be concentrated upon the physiography, visible bedrock geology, and the overburden. The important objectives are: To determine the presence or absence of buried valleys or gorges; to make an approximate estimate of the probable depth to rock; to recognize the rocks, with especial attention to those which are weak or soluble; and to understand in a general way the geological structure, with special attention to faults, zones of weakness, and possible sliding planes. This stage makes the greatest demand upon experience, and a very broad knowledge of geological conditions and engineering requirements is necessary. Such a reconnaissance investigation cannot give the detailed information necessary for design, but it should make possible the choice of the site with the most favorable foundation conditions and should make known the general type, and difficulty, of the foundation problems. Such an investigation can make reasonably certain that the project is economically feasible. In this stage great savings in time and money are possible by discarding unfavorable sites before expensive and time-consuming surveys and studies have been made. Greater use of experienced engineering geologists at this stage is desirable.

The second stage requires a very thorough geological examination of the dam site and vicinity and a thorough subsurface investigation of the site to ascertain whether formations are continuous and homogeneous, or are lenticular, or vary in character, and to work out the details of geological structure and their relation to the dam in order to present to the engineers all pertinent facts about the formations and their structure and to answer all questions of the engineers. This stage of the investigation should give the information needed in the final design of the dam. The engineering geologist is used most in this stage.

After the work has passed beyond the second stage, and the geologist has satisfied himself and the engineers that the natural foundation conditions are such that a safe and satisfactory dam can be built at the site, it is often assumed by engineers that, unless obvious trouble arises, a geologist is no longer needed. However, frequent geological observations during the opening up of foundation excavations will insure that geological details are properly provided for, and may make possible a saving of excavation and concrete. The forecasts made by the geologist during the earlier stages cannot, from the nature of the problem, be 100% correct, and it is very important, therefore, to recognize promptly any variations from forecasted conditions and decide whether they affect the

plans for the dam. Therefore, the continued cooperation of geologist and engineer until the foundations are completely excavated and examined, and all the problems met, will prove most efficient and satisfactory.

Methods of Geological Investigation of Dam Sites

In the preliminary investigation to select the best site and to determine the feasibility of the project, much information can be obtained on many problems, such as the location of buried gorges or concealed faults, by a physiographic study of the area. Unless the geological and physiographic history of the valley is understood, there is danger that some important facts will escape discovery by even the most elaborate drilling program. In poorly mapped areas, aerial observation or the study of aerial photographs with the stereoscope may be very helpful. Where subsurface information is necessary, much drilling may be saved by using geophysical methods. The seismic method is especially suitable for detecting buried gorges since it gives a profile of the bedrock surface. Finally, when the exact site has been chosen and it is desired to develop the details of the site, test drilling will be necessary; but the geological and geophysical investigations will have made it possible to locate the borings strategically so that the necessary information can be obtained by a minimum number of borings. Such a program utilizes the rapid and less expensive procedures of geology and geophysics to keep to a minimum the more expensive and slower method of drilling.

In the second stage of the investigation—that is, the thorough study of the dam site—the geologist's first step is geological mapping of the site and vicinity, the area depending upon the complexity of the geology and other factors. This involves a very careful study to detect all features that may affect the dam, such as zones of depth, weathering, and soluble rocks, and their relation to the dam. If soluble rocks are present, every means must be used to locate underground solution channels and caves. A thorough study of ground-water levels and movements will be useful in estimating seepage around the ends of the dam. Location of test borings with regard to the geological structures may make possible a saving in number and footage of borings and a consequent saving in cost.

The geologist must use every available tool and allied science in solving the complicated problems. In addition to the strictly geological methods, he will need to obtain subsurface information by core borings and test pits and may use shafts and tunnels. Bearing tests and other field tests upon the rock in place may be desirable. Wherever there is question of leakage through the rocks under or around the dam, a study of the permeability of the rocks by means of water tests on drill holes becomes important. In addition to these many field methods, some of the following laboratory tests and procedures may be necessary, depending upon the rocks and structures: Determination of true and apparent specific gravity, porosity, and moisture content; solubility and slaking tests; petrographic examination; chemical analyses; consolidation tests on specimens of shale; compression and shearing tests; and determination of modulus of elasticity. Not all of the field and laboratory tests are needed at any one dam site, and the use of all of them promiscuously would be a

serious waste of time and money. New methods of value in investigations of dam sites have been developed in recent years; but further development and better understanding of certain problems are desirable and will aid in making more practicable thorough investigations of dam sites.

Core borings have long been one of the most useful tools for investigating the rock at dam sites, and it is sometimes helpful, in studying complex geological structures, to record the boring results on three-dimensional models. A recent development of great value is the use of large drill holes from 2 to 3 ft in diameter into which the geologist can descend and study the section of the foundation in its natural conditions and detect faults and zones of weakness. One of the first uses of these holes was at the Prettyboy Dam, in Maryland, in 1931, but they had previously been used in quarries.[13] Various devices have been tried for locating open cracks in smaller drill holes. A periscope was used at Norris Dam, in Tennessee, and at Conchas Dam, in New Mexico, and a feeler which indicates any recesses in the walls of the holes was used also at Norris Dam.[14]

Geophysical prospecting may be used in preliminary investigation and in some cases in the more detailed study of dam sites. The electrical method was used first, but the seismic method has been used at many sites.[14a] Both the seismic and electrical methods have been used successfully under water.[15] Submarine acoustic methods have been used to detect leaks from reservoirs. Geophysical methods were first applied to study of dam sites at the site of the Fifteen Mile Falls Dam, in New Hampshire, to determine the depth to bedrock.[16] The electrical method has been used successfully to study geologic structures and locate faults and decomposed zones.[17] There are several techniques, some of which are described in the articles referred to and in others.[18]

A modification of electrical prospecting known as "electrical coring" has been used extensively in the oil fields and has more recently been applied to dam site investigation.[19] In core drilling it is often impossible to obtain core

[13] "Weathering and Albitization of the Wissahickon Schist at the Prettyboy Dam, Baltimore County, Maryland," by Joseph T. Singewald, Jr., *Bulletin*, Geological Soc. of America, Vol. 43, 1932, p. 450.

[14] "Thousands of Holes Grouted Under Norris Dam," *Engineering News-Record*, November 21, 1935, p. 700; also "Unique Devices Developed to Aid Dam Foundation Grouting," *Engineering News-Record*, August 8, 1935, p. 191.

[14a] "Seismic Refraction Methods as Applied to Shallow Overburdens," by F. L. Partlo and Jerry H. Service, *Transactions*, A. I. M. E., Vol. 110, 1934, pp. 473–492; "Practical Seismology and Seismic Prospecting," by L. D. Leet, D. Appleton-Century Co., Inc., pp. 347–399.

[15] "Electrical Exploration of Water Covered Areas," by C. and M. Schlumberger and E. G. Leonardon, *Contribution No. 71*, A. I. M. E., 1934; "Geophysical Prospecting, Mining and Metallurgy," by Sherwin F. Kelly, January, 1938, pp. 15–17; and "Étude Géophysique Sous-Marine Exécutée dans le Port d'Alger," by Conrad Schlumberger and Pierre J.-M. Renaud, *Annales des Ponts et Chaussées*, IV, 1933.

[16] "Electrical Prospecting Applied to Foundation Problems," by Irving B. Crosby and E. G. Leonardon, *Technical Paper No. 131*, A. I. M. E., 1928; also "Electrical Subsoil Exploration and the Civil Engineer," by Irving B. Crosby and Sherwin F. Kelly, *Engineering News-Record*, February 14, 1929, pp. 270–273.

[17] "Some Application of Potential Methods to Structural Studies," by E. G. Leonardon and Sherwin F. Kelly, *Technical Publication No. 115*, A. I. M. E.; also "Application des Méthodes de Prospection Électrique à l'étude des fondations de Hauts Barrages et des Ouvrages Annexes," by Maurice Lugeon and Conrad Schlumberger, *Le Génie Civil*, August 6, 1932, pp. 134–137.

[18] "Application of Electrical Prospecting to the Study of Dam Sites," by C. and M. Schlumberger, 2d Cong. on Large Dams, Vol. IV, pp. 67–80, Washington, D. C., 1936; "Topographical Study of a Hidden Bedrock Surface by Resistivity Measurements," by E. G. Leonardon, *Engineering Journal* (Canada), June, 1931; and "Electrical Exploration Applied to Geological Problems in Civil Engineering," by E. G. Leonardon, *Technical Publications No. 407*, A. I. M. E., 1931.

[19] "Electrical Coring; a Method of Determining Bottom-Hole Data by Electrical Measurements," by C. and M. Schlumberger and E. G. Leonardon, *Technical Publications No. 462*, A. I. M. E., New York, 1932; also "A New Contribution to Sub-Surface Studies by Means of Electrical Measurements in Drill Holes," by the same authors, *Technical Paper No. 503*, A. I. M. E., New York, 1933.

at the most critical places, but "electrical coring" gives information about the rock at those places where core cannot be obtained. If the method is correlated with a few actual cores from core drill holes, it is possible to identify the same strata in holes drilled without coring. Information is also obtained about the porosity of the rocks, liquid pressure in the rocks, and strike and dip of the rocks. If there is a fractured zone from which it was impossible to obtain core, the nature of this zone can be determined.

In addition to the field examination and the field tests, many laboratory tests upon specimens of the foundation materials may be necessary as part of the geological investigation.

Direct shearing tests made on rocks in any of the direct shearing devices do not give the true shearing resistance since progressive failure takes place due to concentrated stress. The approximate shearing resistance can be computed from unconfined compression tests on cylinders with length at least twice the diameter. The true shearing resistance can be computed from properly conducted triaxial compression tests on cylinders, with length at least twice the diameter, using different confining pressures.[20] Attempts to estimate from triaxial compression tests the actual shearing resistance of rock in the ground might give too favorable results because it is impossible to reproduce, in the laboratory, the effect of cracks in the bedrock. The results of shearing and compression tests should be interpreted by the geologist after observation of the rocks under load in Nature and after study of conditions as they will exist in the foundations.

Geology of Dam Sites

The problems of masonry dams on rock vary with the type of rock, and the problems of each broad rock type will be discussed separately and illustrated by examples of successful dams and of failures. However, there are problems dependent upon structure or other geological characteristics, independent of the rock type, and these will be discussed first. The first problem is the selection of the dam site with the best bedrock profile, and the most difficult part of this problem is to detect any buried gorge in the bedrock surface, either at or near the dam site, which might be missed in the preliminary test drilling. Such a gorge was found at one site, between drill holes spaced 50 ft apart.

Buried valleys or gorges are most common in glaciated regions where the drainage was disarranged by the ice sheet. The postglacial streams have often cut down upon rock ridges, ignoring their old valleys now filled with glacial drift. This is generally the explanation in the glaciated regions for a stream with rapids or falls in a narrow rock gorge with wider valleys above and below. Each type of buried gorge or valley is associated with a definite geological history and is indicated by physiographic features which serve as valuable clues to the geologist experienced in their interpretation. Sometimes these features will show up well from the air or in aerial photographs.

[20] "The Shearing Resistance of Soils," by L. Jurgenson, *Journal*, Boston Soc. of Civ. Engrs., July, 1934, pp. 242–275; "Versuche ueber Festigkeitseigenschaften von Sand in dreiachsigen Spannungszustand Wasserwirtschaft und Technik," by W. Bernatzik, 1935, p. 184; "Report on an Apparatus for Consummate Investigation of Mechanical Properties of Soils," by W. Kjellman, *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., June, 1936, Vol. 3, pp. 16–20; and "Ein Grundgesetz der Tommechanik und sein experimenteller Beweis," by L. Rendulic, *Der Bauingenieur*, 1937, Nos. 31–32.

Buried valleys may underlie the reservoir site and lead from it at points far from the dam site. They will not directly affect the dam, but they may cause serious leakage from the reservoir. The detection of possible leakage from the reservoir, except at the dam site, whether through buried valleys, through faulted or crushed zones, through solution channels in soluble rocks, through lava tunnels, or through porous rocks, presents important and difficult problems which cannot be discussed at length in this paper. They are problems, however, that are especially suited to geological and physiographic study, and in which geophysical methods and ground-water studies may be very valuable. Leakage at the dam site or elsewhere in the reservoir basin, although it may not endanger the dam, may render a good dam useless by preventing storage of water in the reservoir. The seriousness of any given amount of leakage is governed by the use for which the reservoir is intended. Decision as to whether leakage will continue to remain harmless or become serious is a geological problem requiring careful study and great experience.

Among the geological structures which may be of importance in relation to dams are: Faults, crush zones, joints and zones of close jointing, bedding seams, sharp folds, zones of deep weathering, zones of alteration, solution channels and cavities, and lava tunnels. A fault is a fracture along which there has been differential movement, and its relation to dams may be important or relatively unimportant depending upon its condition. Recent faults raise the question of future movement with accompanying earthquake action. A fault may be an open fissure, or it may be filled with gouge or crushed rock. In either case it is important in regard to leakage as well as in regard to structural strength. The presence of gouge increases the difficulty of grouting the fracture. Although a number of dams have been built across faults, there is no known record of a masonry dam across a live fault during an earthquake. The Crystal Springs concrete gravity dam, of the San Francisco water supply, is less than a quarter of a mile from, and parallel to, the San Andreas fault on which the earthquake of 1906 occurred. The dam was not injured by the earthquake, but it was not intersected by the fault. The Rodriguez, Ambursen-type dam on the Tiajuana River, in Mexico, was built across a dead fault which intersected the axis of the dam under the stream bed.[21]

Morris Dam in San Gabriel Canyon, California, is a gravity, concrete dam across a fault. Geologic evidence in the form of undisturbed river gravels indicated that there had been no movement on the fault for thousands of years but provision was made for possible future movements by the construction of an earthquake joint. Examination of striæ on the walls of the fault indicated that movement had always been in the same direction. An open joint has been provided along the trace of the fault between blocks 8 and 9, as described in 1934 by Samuel B. Morris, M. Am. Soc. C. E., and Cecil E. Pearce, Assoc. M. Am. Soc. C. E.[22] The joint will provide for differential movement of 6.55 ft along the fault.

---

[21] "Unique Cutoff Construction and Arched Foundation Features of Rodriguez Dam," *Engineering News-Record*, October 16, 1930, pp. 600–604; also "Foundation Treatment at Rodriguez Dam," by Charles P. Williams, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 99 (1934), pp. 295–305.

[22] "A Concrete Gravity Dam for a Faulted Mountainous Area," by Samuel B. Morris and Cecil E. Pearce, *Engineering News-Record*, December 27, 1934, pp. 823–827.

Joints are fractures along which there has not been appreciable movement. They are generally nearly tight at depth but are usually somewhat open near the surface. They intersect practically all rocks and are often in systems, with the joints of each system approximately parallel to each other. Joints cut up the rock into blocks of various sizes, decrease the structural strength of the rock, and are important in regard to seepage. The relation of the direction of the principal joints to the axis of the dam is important in connection with foundation stability, seepage, and grouting. Consequently, it is important for the geologist to determine the relation of the joints and other fractures to the dam and their effect upon it in regard to stability and seepage.

The different beds of stratified rock are separated by bedding planes. The rock may be continuous across these planes, but they are zones of weakness. In some cases they are shale seams; occasionally they are open cracks, in which case they are important in regard to seepage. Folding of the strata may have tended to open bedding seams and joints, especially along the crests of anticlines, and may have formed transverse joints. Joint cracks and bedding seams are especially important in regard to leakage under or around the dam.

The resistance of rocks to erosion is important in regard to the effect of overflow on the rocks below the spillway. Their resistance depends partly on their strength and hardness but even more on their structure. Closely jointed and thin-bedded rocks are especially susceptible to erosion since the rushing water enters joint cracks and bedding seams, exerts hydrostatic pressure, and pries off slabs and loosens blocks. The actual abrasion of flowing water, even when armed with sand and gravel, is relatively unimportant compared to its quarrying action where the rock is fractured.

The increasing height of dams emphasizes the importance of the elastic properties of rocks and of the strain conditions existing in the rocks. In some cases tests upon specimens of rock, for the modulus of elasticity, and upon the rock in place (as in tunnels), for residual strain, should receive attention in order that the design of the dam may meet the conditions imposed by the different elastic properties and strain conditions of the rocks and of the concrete. Expansion of rocks upon relief of stress is a well-known occurrence in quarries, and residual strain may be expected in rocks in deep canyons. The greatest contrast in the elastic properties of rocks and concrete probably occurs with some uncemented shales, and in such cases the rock will deform under load differently from the concrete. Excessive deformation of the foundation rock may injure grout cutoffs and the bond between rock and concrete. The practical importance of the elastic properties of rocks is greatest in the foundations of high gravity dams and the abutments of arch dams.

Arch dams make especially severe demands upon the abutments since the rock in these must be strong enough to withstand the thrust of the arch. The structure of the rock is unusually important here, and rock which in itself is amply strong may be intersected by joints or faults in such a way that the mass as a whole cannot offer the necessary resistance to the thrust. Therefore, the exact condition of the rock and of its structure must be determined and the effects of the new stresses upon it must be studied. The effects of deep weathering and alteration must receive careful study also. These effects tend

to be more serious in abutments outside the glaciated regions. The possibility of earthquakes, and the presence of any faults upon which movement might be renewed, become of unusual importance with this type of dam.

Dams on Shale

The term shale is used in the broad sense in this paper, and it is not confined to those that have a well-developed lamination or thin-bedded stratification, but includes those unlaminated, argillaceous rocks with no apparent stratification which may appropriately be called siltstones or mudstones. Shales are formed by the compaction of clay and silt with, or without, cementation. Depending upon the degree of compaction and cementation, they may vary from a very soft, weak rock, little different from clay, to a strong, weather-resisting rock. They may be divided into shales that have not been cemented, but have been compacted only, and shales that have been cemented in addition to compaction. The practical problems of dams on shales depend upon the type of shale and, for the laminated shales, they are: Sliding, uplift, strength of the shale, seepage through fractures, and the preparation of foundations. With uncemented shales the important problems are: Strength of the shale, sliding, settlement due to additional compaction, elastic deformation, prevention of disintegration, and preparation of foundations. The uncemented shales weather very rapidly due to the fact that they dry out and crack when exposed to the air and disintegrate when wet again. The cemented shales weather more slowly and do not disintegrate with alternate drying and wetting, but there are often poorly cemented strata which weather rapidly.

Shale probably requires greater care and more precautions in the preparation of the foundation than any other rock. If the shale is laminated, and especially if the stratification is flat, care must be used to remove all weathered and loose, or partly loose, slabs without causing an unnecessary amount of excavation. Blasting may often injure the shale and require much deeper excavation than would otherwise be necessary. If the shale is uncemented, great care must be exercised to prevent drying and disintegration when exposed. If the surface of an uncemented shale is allowed to dry for even a short time before concrete is placed upon it, the water of the concrete causes the surface of the shale to slake and form a permanent layer of mud between the concrete and the soft clay. This condition can be avoided by reducing the time of exposure, and by coating the freshly exposed surface of the shale with an asphaltic or other protective coating. Such protective coatings have been used with satisfaction at the Muskingum Dams, in Ohio, Fort Peck Dam, in Montana, Conchas Dam, and elsewhere. Experience and experiments have proved that the bond between the concrete and uncemented shales is improved by coating the shale with a waterproof preparation.

Shales are relatively watertight since joints are often not well developed in them and the bedding planes are generally tight. In the case of uncemented shales, existing cracks tend to be closed by plastic flow due to the weight of the dam. Open fractures are more likely to occur in the cemented shales and, since some of the beds are often less well cemented and are subject to disintegration and erosion, the closing of all cracks by grouting is important. In lam-

inated shales, and in fact in all stratified formations where there is a possibility of open bedding seams, it is important to avoid excessive grout pressures since pressure, whether from grout or test water, in a partly open bedding seam tends to lift the overlying rock and open the seam farther. Cases of actual measured uplift of structures have been observed. Obviously, if the pressure is restricted to the net weight of the overlying material, there can be no uplift and this is a safe rule. A geological investigation of the condition of flat seams should give information on this point. Grout pressure sufficient to cause uplift may actually injure the foundations as well as waste grout.

The compressive strength of shale varies greatly, depending upon the amount of compaction and cementation. Unconfined compression tests on cylinders of an uncemented shale from Texas showed compressive strengths ranging from 156 to 785 lb per sq in., and similar tests on partly cemented shales from the "coal measures" of West Virginia gave results varying from 3,036 to 4,925 lb per sq in. These values show the great variation in crushing strength of different shales and the necessity of testing the different beds of each specific shale. The modulus of elasticity for an uncemented shale in Texas varied from 49,800 to 207,000 lb per sq in. at 100 lb per sq in. load, which is far below the value for concrete.

Resistance to sliding is of utmost importance with dams on laminated shale. The irregularities of the contact between concrete and shale can be made sufficient to prevent sliding of the concrete upon the shale. The problem, therefore, becomes one of the sliding of one bed of shale upon another, which depends upon the shearing resistance of the shale along the bedding planes, or the coefficient of friction if the bedding plane is a complete fracture. Geologic examination should indicate the extent to which bedding planes are actual fractures. Then it can be seen whether the problem is one of shearing or of friction of shale upon shale. The inclination of the bedding has a most important effect upon the resistance to sliding since, if the bedding is flat or is inclined slightly downstream, resistance to sliding will be at a minimum; but if the bedding is inclined upstream, resistance to sliding will be increased. If the dip is even moderately steep, sliding will involve shearing the shale across the beds, which will offer much greater resistance than shearing along the bedding planes. The deeper the base of the dam is set into the shale, the greater will be the resistance to sliding, since this will throw the potential shear planes deeper into the shale and necessitate shearing across the bedding. This may be accomplished without deepening the main part of the foundation excavation by constructing a reinforced anchoring wall as was done at Lake Lynn Dam on the Cheat River, in West Virginia, or by sinking reinforced concrete dowel pins into the shale as was done at the spillway gate structure at Fort Peck Dam.

Uncemented shales have undergone deformation due to the irregular removal of load as in the erosion of a canyon. This may have resulted in the formation of numerous small shear fractures which weaken the shale, and this possibility must be considered in the geological investigation. Uncemented shale, like clay, is subject to additional compaction resulting in squeezing out water when it is subjected to additional load. Therefore, if a dam is built

upon uncemented shale, the shale will compact and the dam will settle. This settlement may be slight or it may be serious, depending upon the thickness of the shale, its degree of compaction, and the amount of additional load. The settlement will extend over a period of years, and its rate will depend upon the thickness of the shale and its permeability. The rate and extent of additional compaction can be determined from a "consolidation test" and from this and geological knowledge of the shale an approximate estimate of the amount and rate of settlement can be made. The fact that the shale may have formerly been subjected to a much greater load and may have had a higher degree of compaction does not prevent additional settlement when additional load is placed upon it. Several examples of concrete dams on shale will be described briefly to illustrate the points discussed.

Conchas Dam.—Built by the U. S. Army Engineers on the South Canadian River in eastern New Mexico, Conchas Dam is a good example of a dam underlain by both uncemented shale and sandstone.[23] The shale is a typical unlaminated, uncemented siltstone. Only two problems will be discussed herein. The north end of the dam, 90 ft high, rests upon a layer of sandstone 20 ft thick, underlain by 70 ft of shale which outcrops in the canyon wall. This part of the dam is supported by the shale, and the principal practical problems are shearing strength of the shale and settlement of the dam due to additional compaction of the shale under increased load. Analysis has shown that the highest stresses in the shale will be in the canyon wall at the heel and toe of the dam and that they will be less than the shearing resistance of the shale as determined by tests on specimens. These tests, however, could not take into consideration the effect of the numerous fractures in the shale; but this was checked by observations on the shale in the canyon walls where it is under load greater than that in the dam.

This shale was formerly under a greater load and was compacted more than at present, but it is still subject to greater compaction under increased load. It was computed that the load of the dam will cause a small, slow settlement which, under extreme conditions, might eventually be a few inches, but which will probably be much less. Since the higher, central part of the dam rests on sandstone below the shale, it will not be subject to this settlement and there will be differential movement between the parts of the dam which is provided for by a special joint.[24]

Tygart Dam.—Tygart Dam, 235 ft high, was completed in 1939 by the U. S. Army Engineers on the Tygart River in West Virginia. It is another high dam on sandstone and shales. The shales include both laminated and massive uncemented varieties, and partly cemented, laminated, and thick-bedded shales. The river-bed section of the dam rests upon a thick bed of sandstone, but the abutments are underlain by alternating beds of sandstone, sandy shale, thick-bedded carbonaceous shale, laminated shale, a massive uncemented shale, and thin beds of coal.

The principal problems were: Adequate foundations, prevention of sliding, prevention of disintegration during construction, and seepage. It was pos-

[23] For a more complete discussion of the geology and foundation problems of Conchas Dam, see "Engineering Geology Problems at Conchas Dam, New Mexico," by Irving B. Crosby, *Proceedings*, Am. Soc. C. E., January, 1939, pp. 29–47.

[24] "Dam Building on Difficult Rock," *Engineering News-Record*, June 9, 1938, pp. 808–809.

sible to have the foundations of each monolith on a bed of sandstone or sandy shale, thus avoiding the problem of foundations directly upon uncemented shale. Disintegration of the uncemented shales was a serious problem which was met principally by not excavating to the final lines until it was time to place concrete. Adequate resistance to sliding of the abutments was obtained by carrying the dam farther into the hillsides than would have been needed with stronger, more massive rocks. The bed of sandstone under the river section of the dam was underlain by a great thickness of shales with some thin coal seams. The shales were intersected by joint cracks and the coal seams were fractured. A grout curtain, 150 ft deep, was constructed and at first high grouting pressures were used; but the holes took excessive quantities of grout and it was impossible to grout some of them to refusal. It appeared possible that the high pressure was opening some of the bedding seams and the grout pressure was reduced, with the result that there was no further difficulty.

Lake Lynn Dam.—Lake Lynn Dam, formerly called State Line Dam, on the Cheat River in West Virginia, near the Pennsylvania state line, has been in use since 1928. It is a good example of a concrete gravity dam, 100 ft high, resting almost entirely on shales, including the soft uncemented variety. It does not have the advantage of a thick bed of sandstone for the foundation of the high part of the dam as was the case with Conchas and Tygart dams, but, by means of special precautions, a satisfactory and safe dam was obtained. The high river section of the dam rests upon 10 to 15 ft of hard, gray, partly cemented, laminated shale. Beneath this stratum are beds of limestone, calcareous shale, coal, and uncemented shales. The principal problems were sliding and seepage. Sliding might take place in the laminated shales and seepage in the fractured coal. A very intensive investigation and analysis of conditions were made and they were met by the following procedures: The dam was keyed into the shale by a reinforced anchoring wall, extending 10 ft below the general foundation at the heel, and by a toe extending 7 ft below the general foundation. These throw the shearing stresses deep into the foundation shales and insure that sliding would involve shearing the shale across the bedding. The toe abuts against a mass of shale that is protected from erosion by a concrete apron. From the bottom of the anchoring wall a concrete cutoff extends down through the coal seam, and a grout curtain extends below that. The grout pressures used were generally 80 lb per sq in. and, although the rocks are laminated with flat bedding seams, there does not appear to have been any uplift. This dam site had several difficult conditions which were met successfully.

Failure of Dams on Shale

A number of dams on shale have failed by sliding and of these Ohio River Dam No. 26 is a typical example. The part which failed by sliding was of the Chanoine wicket type with a total height of about 20 ft. Apparently, it slid on a bedding plane in the shale about 0.2 ft below the base of the concrete.[25] The shale was laminated, soft, and poorly cemented, with a surface greasy to the touch. The toe of the dam did not abut against rock, and failure was caused by sliding on bedding seams along which resistance was low and had

[25] *Professional Memoirs*, U. S. Engr. Dept., Vol. 5, May–June, 1913, pp. 315–322.

probably been further reduced by uplift. There does not appear to have been anything unusual about this failure and it is not necessary to suppose that there was an unpredictable system of joints.[25a] There is no record of any geological examination of this site having been made before the dam was built.

Dams on Sandstone and Conglomerate

Sandstone is a rock composed of grains of rock or minerals of sand size. It grades downward into shale and upward into conglomerate. The grains are usually cemented together by calcareous, siliceous, ferruginous, or argillaceous cement but cement may be practically lacking. Calcareous sandstone grades into limestone, and argillaceous sandstone grades into shale. Sandstone may be thick or thin bedded, the thickness of individual beds varying from several feet down to a fraction of an inch. The thin-bedded varieties may have shaly characteristics. Sandstones are more or less porous and permeable. Conglomerate is consolidated gravel, the pebbles of which are cemented together.

The principal practical problems of masonry dams on sandstone are seepage, uplift, and sliding on bedding seams. Poorly cemented argillaceous sandstones or conglomerates may disintegrate with changed conditions and cause serious foundation problems and may render a site unfit for a masonry dam. Flat-bedded sandstones are especially susceptible to erosion at the toe of overfall dams and may require protection. Sandstones are intersected by joints and bedding seams and are also pervious, with the result that water can travel through pores, as well as through fractures, and hydrostatic pressure can diffuse throughout the rock. Therefore, problems of seepage and uplift are unusually important in sandstones.

There is great variation in the rate of weathering of sandstones due principally to the varying resistance of the different cements. Siliceous sandstones are the most resistant, and argillaceous sandstones the least so. The solution of the calcareous cement from a sandstone is believed to be generally too slow to be serious during the life of a dam, but research is desirable on this point. Shaly beds and partings weather faster than the more massive sandstones and this may require more extensive foundation excavation.

The crushing strength of sandstones and conglomerates varies greatly, and each formation must be tested separately. A series of unconfined compression tests on sandstones gave results varying from 4,920 to 33,350 lb per sq in.[26] A calcareous sandstone crushed at 18,810 lb per sq in. and a series of tests upon ferruginous sandstones gave results varying from 7,272 to 18,350 lb per sq in.[27] Some argillaceous sandstones and some of the sandstones with little or no cement are much weaker than those for which figures have been given. The argillaceous conglomerate from St. Francis Dam had a strength of only 523 lb per sq in., but the siliceous Roxbury conglomerate had a compressive strength of 17,360 lb per sq in.

[25a] "Effect of Minor Geologic Details on the Safety of Dams," by Charles Terzaghi, M. Am. Soc. C. E., *Technical Publications No. 215*, A. I. M. E., New York, 1929, p. 31.

[26] "Physical Properties of Typical American Rocks," by John H. Griffith, *Bulletin No. 131*, Iowa Eng. Experiment Station, 1937, pp. 10–11.

[27] "A Descriptive Catalogue of the Building Stones of Boston and Vicinity," by W. O. Crosby and G. F. Loughlin, *Technology Quarterly*, Vol. 17, 1904, pp. 165–185.

Sliding of a dam on sandstone takes place by shearing along bedding seams and is dependent upon the shearing resistance of the sandstone parallel to the bedding. The shearing resistance when wet is usually less than when dry, and alternate drying and wetting further reduces the shearing resistance of some sandstones, especially the shaly ones. The most important factor in regard to sliding is the presence and relation to the dam of shaly seams or open bedding cracks.

Conchas Dam.—Conchas Dam has already been described as an example of a dam on uncemented shale; but the highest part of the dam rests upon sandstone containing water under artesian pressure, and is an excellent example of dams on this rock. Since this sandstone is porous and permeable, and is intersected by joint cracks and bedding seams, the problems of seepage and uplift were important.[23]

Tygart Dam.—Tygart Dam has already been described as an example of a high dam upon shales, but the highest section rests upon a bed of sandstone. The layer of sandstone upon which the high section of the dam rests is thick bedded but is cut by joint cracks into irregular blocks. In order to make it more nearly a continuous layer "consolidation grouting" with shallow holes on 20-ft centers was resorted to over the entire base of the dam. The most serious problem was the prevention of sliding. The dam was bonded to the rough, irregular surface of the sandstone with the result that sliding would involve shearing across the bedding of this thick-bedded, strong rock, which is at least 14 ft thick. Since this sandstone is stronger than concrete, sliding is practically impossible as long as the bed of sandstone immediately downstream from the dam is kept intact, which is provided for by a stilling basin.

Failure of Dams on Sandstone and Conglomerate

The greatest danger of failure of dams upon the siliceous, calcareous, or ferruginous sandstones is from sliding. These rocks are always more or less stratified, and sometimes the bedding seams are definite cracks. At Austin, Pa., a concrete gravity dam, 50 ft high on sandstone with shale bedding seams, failed by sliding. Water seeping under the dam probably caused uplift, and the dam slid on a shale bedding seam just below the foundations. No geological examination of the site had been made before the dam was built.

With dams upon argillaceous sandstone and conglomerate there is the added danger of weakness of the rock and the effect of water upon it. The St. Francis Dam was a good example of the failure of a dam on argillaceous conglomerate. The western part of the St. Francis Dam rested upon such a conglomerate, which consists of sand and pebbles of hard rock cemented with clay. It is essentially an uncemented mudstone containing pebbles, and has a very low compressive strength.[28] The reports made after the failure agreed in assigning this weak "conglomerate" as an important cause of failure.[29] This rock disintegrates when placed in water and illustrates the danger of such argillaceous sandstones or conglomerates in foundations. Since there

[28] "Essential Facts Concerning the Failure of the St. Francis Dam," by the late Louis C. Hill, Past-President, Am. Soc. C. E., H. W. Dennis, and F. H. Fowler, Members, Am. Soc. C. E., Report of Committee of Board of Direction, Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., Vol. LV, October, 1929, p. 2156.

[29] *Loc. cit.*, pp. 2147–2163.

was no other cement than clay, and since this softened when wet, the rock lost what strength it had in the dry condition and the dam failed. No geological investigation was made of the site before the dam was built.

Dams on Limestone and Other Soluble Rocks

Limestone is the most common soluble rock; but in this class are also marble, dolomite, gypsum, and rock salt. Limestone is composed of grains or crystals of calcite, or of fragments of calcium carbonate, as with shell limestone or coquina. The rock dolomite is a magnesian limestone composed principally of the mineral dolomite. Marble is metamorphosed, crystalline limestone.

The principal practical problems of dams on limestone and other soluble stratified rocks are leakage and sliding on shaly bedding seams. Limestones are intersected by joints and bedding seams in much the same manner as sandstones are, and jointing is often developed in a very regular manner. Joints and bedding cracks, however, are of unique importance in limestones due to the fact that the rock is soluble and that water passing through the cracks enlarges them and may form extensive caverns. Limestone is more soluble than dolomite but is much less soluble than gypsum. Geologically, limestone dissolves rapidly, but from the human viewpoint its solution is very slow and it is not believed that it is sufficiently rapid to enlarge joints and other fractures seriously during the life of a dam. With gypsum the case is different and its solution is sufficiently rapid to be serious during the life of a dam. The formation of caves and underground drainage systems depends upon many factors such as: The frequency, regularity, continuity, and openness of joints and bedding seams; the degree of solubility of the particular limestone; the presence of more or less soluble layers; and the structure and the stage of physiographic development of the region. These and other factors must be studied carefully by the geologist in order that the nature and extent of solution channels can be determined. Subterranean drainage systems have divides which may or may not coincide with surface drainage divides. Underground drainage divides must be located because their relation to the dam is of great importance, and study of ground-water levels is of assistance in this problem. Although limestone presents many serious foundation difficulties and uncertainties, many successful dams have been built upon it. Many attempts have been made, with varying degrees of success, to make cavernous limestone foundations tight by grouting. This subject has been treated adequately in previous papers.[30]

Limestones vary greatly in strength, a dense variety having a crushing strength of more than 40,000 lb per sq in. and coquina, shell limestone, a crushing strength of 220 lb per sq in. Many limestones crush at 10,000 to 20,000 lb per sq in. Sliding of dams on limestone will involve shearing of the limestone if the dam has been properly bonded to the rock. Most limestones have a high shearing resistance transverse to the bedding, and sliding is possible only along bedding seams. Therefore, if the foundation in such rocks is so designed that sliding would involve shearing across the bedding, danger of

---

[30] "Improving Foundation Rock for Dams," by James B. Hays, M. Am. Soc. C. E., *Civil Engineering*, May, 1939, pp. 309–312; also "Corewall Grouting at Chickamauga Dam," by James B. Hays, *Engineering News-Record*, April 27, 1939, pp. 59–60.

sliding is generally eliminated. If the limestone is thin bedded or has shaly partings, the possibility of sliding is increased.

Wilson Dam.—Situated at Muscle Shoals, Ala., on the Tennessee River, Wilson Dam is a good example of a successful dam built on limestone. The problems involved prevention of seepage and leakage around the dam through caverns and prevention of sliding and erosion below the dam. A thorough geological investigation was made. Under the river are alternating thick beds of chert and limestone. Above these, and forming most of the abutments, is thin-bedded, interlaminated chert and limestone. The beds dip gently to the southwest—that is, downstream and toward the left bank.

Numerous small caves occur in the south bluff; there are larger caves in the vicinity and large springs at Tuscumbia, 6 miles downstream. There had been some fear that these springs were connected with the river and that there might be serious leakage when the reservoir was full. A very thorough study was made of caves, joints, and springs in the vicinity. It was shown that the large springs were the outlets of the underground drainage system of a large area south of the dam, that there was an area of high ground water between this area and the dam, and that consequently there could be no connection between the springs and the river at or above the dam. It was also shown that the rock in the river bed could be made tight by reasonable grouting and that the only place where special treatment was necessary was at the south abutment. Two tunnels were driven into this rock and all open seams were cleaned out. Extensive grouting was done, and the tunnels were filled with concrete. The rocks under the river were susceptible of uplift from excessive grout pressure; but this was avoided by restricting the pressure to 30 lb per sq in.

Hales Bar Dam.—Hales Bar Dam on the Tennessee River is an excellent example of the difficulties that may arise when a dam is built on limestone without adequate investigation. In so far as can be learned, no geological investigation was made of the site before the dam was built, but several investigations have been made since to learn what could be done to remedy conditions. The dam is on limestone which is cavernous, and great difficulty was encountered in unwatering the cofferdams. Much grouting was done and compressed air caissons were used.[31] Since completion there have been very serious leaks under the dam which could not be stopped by any method until asphalt grouting was tried. This was partly successful and reduced the leakage. It is reported that $10,000,000 was spent in attempting to control the leakage. It is certain that if an adequate geological investigation had been made before the dam was built these difficulties could have been foreseen and it is probable that a better site could have been found.

Failure of Dams on Limestone

A recent example of serious leakage under a dam on limestone, where investigation of the condition of the limestone before construction had been inadequate, is the Ontelaunee Dam in Pennsylvania. Leakage developed under the core wall of this earth dam shortly after the reservoir was filled, and the

[31] "The Power Development at Hales Bar," by J. A. Switzer, *Resources of Tennessee*, Vol. 2, No. 3, Nashville, 1922, pp. 86–99.

embankment was endangered. The reservoir was drained and the condition was finally remedied by additional grouting.[32]

Some twenty years earlier, part of the masonry wall forming a reservoir at Nashville, Tenn., failed by sliding. The wall rested upon a thin-bedded, flat-lying limestone in which were bedding seams and thin strata of shale. The dam slid on a shale bedding seam 4 ft below the base of the concrete, and 4 ft of rock adhered to the concrete and slid with it. No geological examination of the site had been made before the dam was built.

The Austin Dam on the Colorado River in Texas is a well-known example of a dam which failed by sliding. The limestone has porous beds and shaly seams, and sliding occurred on one of these seams below the base of the concrete, a layer of rock sliding with the dam. The rock at the toe had been removed by erosion, and there was some undermining of the toe of the dam. The dam could slide on a bedding seam, therefore, without shearing across the bedding. No record has been found of any geological investigation having been made before the dam was built.

Dams on Gypsum

Dams on gypsum present even more serious problems than dams on limestone because of the much greater rate of solution of gypsum. There have been a number of cases of very serious leakage under or around dams on gypsum, but there are also some reasonably tight reservoirs in gypsum. If there are no joint cracks or other fractures or openings for water to start through, gypsum may dissolve about the shores of the reservoir without causing any leakage. Apparently, such was the case with the Avalon Reservoir and Willow Lake in New Mexico. McMillan Reservoir, near the Avalon Reservoir, leaked badly, as did also the Honda Reservoir, also in New Mexico. In some cases projected dams on gypsum were abandoned when the conditions were disclosed by geological investigation.

Dams and Reservoirs on Basalt and Other Lavas

Basalt and other lavas cover vast areas in the northwestern part of the United States and in other parts of the world, and therefore many dams have been built upon them. Since some of these lava regions are also semiarid regions, the need of storage reservoirs in them is great. Basalt includes the dark-colored lavas as differentiated from the felsites, or light-colored lavas. Basalt may be a massive, dense rock, it may be intensely fractured, or it may be porous. The felsites are less likely to be intensely fractured, but they may be extremely porous.

These lavas present some of the most difficult problems in dam geology, principally because of the fact that they are generally highly fractured. The problems, therefore, are principally concerned with loss of water, and this may not necessarily occur at the dam site. Sometimes the rocks at the dam site are sufficiently tight, or can be made so, but serious leakage may occur from other parts of the reservoir basin. There are cases in which dams that have

32 "Caverns Under Dam Corewall Set a Nice Repair Problem," by Farley Gannett, M. Am. Soc. C. E., *Engineering News-Record*, April 2, 1936, pp. 492–494.

been entirely satisfactory in themselves have been rendered useless by the fact that severe leaks elsewhere in the reservoir basin made it impossible to store water, and in some cases impossible even to fill the reservoir. The most important problem of dams on basalt is likely to be tightness of the reservoir rather than safety of the dam.

Basalts may be more intensely fractured than most other rocks or they may be massive and relatively free from close jointing. Basalts which formed as thin flows and cooled quickly may have the typical columnar jointing which divides the rock into columns, sometimes hexagonal, of several inches to a few feet in diameter. Each column is surrounded by cracks and since these cracks were formed by shrinkage of the cooling mass they are less likely to be tight at depth than other joint cracks. As a result some basalts permit free movement of water.

Lava tunnels in basalts are a unique type of open passage which is of extreme importance in relation to dam sites and reservoir basins. These are natural tunnels in the lava which may extend for miles and may be 30 ft or more in diameter. They may be open or partly or entirely filled with sand which has washed into them. Those which are known were discovered where the roofs had fallen in, and there are doubtless many which have never been discovered. Obviously, one of these tunnels underlying a reservoir and leading outside it could cause tremendous leakage. Their discovery by drilling only is not practicable in a large reservoir basin and the best method for locating them is by a thorough geological and ground-water investigation with drilling at suspicious points.

Because of the close jointing and the possibility of lava tunnels, basalts are among the most leaky and treacherous rocks with which the dam builder has to deal. Ground-water studies, therefore, are especially important for reservoirs in basalt. If it is found that the ground-water table is far below the reservoir basin, or if there is a deep canyon nearby, the probability is great that there will be serious leakage from the reservoir. However, if it is found that the ground-water table is high and rises from the reservoir basin, it is probable that a dam can be built safely and that the reservoir will be sufficiently tight. In general, basalts and other lavas are very strong rocks, but their strength is determined more by the amount of fracturing than by their mineralogical composition. Therefore, any tests upon specimens must be interpreted in the light of a thorough geological investigation of their structure.

There are many successful dams on lavas and some which have been failures, due to the fact that the reservoirs would not hold water. A few examples will illustrate some of the problems. The two examples of failure happen to be reservoirs formed by earth-fill dams, but since the leaks were not at the dam site they had nothing to do with the type of dam and would have occurred with masonry dams.

American Falls Dam.—At American Falls on the Snake River in Idaho, a concrete gravity dam 87 ft high was completed by the Bureau of Reclamation in 1927. The dam is founded on a bed of relatively massive columnar basalt which is underlain by a bed of obsidian (volcanic glass) about 23 ft thick. This in turn is underlain by a bed of tuff. The dam site is intersected at the western

end by an old fault. The dam is near the crest of the falls, a position which was required by the topography but which tends to increase seepage, due to the possibility of short-circuiting under the dam to the river below the falls. The reservoir basin which is approximately 25 miles long is generally, but not completely, blanketed by relatively impervious, sedimentary lake deposits, clays, and fine sands. Although there has been considerable seepage from the reservoir, it has not interfered with its economically successful use for irrigation storage. The blanket of impervious lake beds on the floor of the reservoir is at least partly responsible for its success.

Jerome Reservoir.—The Jerome Reservoir on the Snake River lava plain about 8 miles northeast of Jerome, Idaho, was formed by two earth-fill dams and was fed by a canal. A layer of volcanic ash covered the bottom of the reservoir but was underlain by fractured basalt. The ground-water level was very low, which should have been a warning. When it was attempted to fill the reservoir, water broke through the layer of ash into the underlying basalt, forming funnel-shaped sinks in the ash. A flow of 2,000 cu ft per sec would not fill the reservoir, which was then abandoned. The contrast between this reservoir and the successful American Falls Reservoir is due to the contrasting geological conditions. Jerome Reservoir was in a dry valley where the ground-water level is very low, whereas American Falls Reservoir is in a river valley which had once been occupied by a lake, and which had a nearly complete blanket of impervious sediments on its floor; also, the ground-water level was higher.

Tumalo Reservoir.—Tumalo Reservoir on the eastern slope of the Cascade Range, in Oregon, is another example of failure similar to the Jerome Reservoir. It was formed by an earth-fill dam across a small canyon. The bottom of the reservoir was covered with volcanic ash underlain by fractured basalt. The water seeped through the ash, carried it into the fissures, and formed open sinks. This reservoir was abandoned.

Dams on Granite and Other Intrusive Igneous Rocks

The practical problems of dams on the granites and on the other intrusive igneous rocks are similar. These rocks include the true granites, syenite, diorites, and gabbros. They were all hard, strong rocks when formed, but they may have been so altered that their present characteristics are unfavorable for foundations. They may be among the best foundation rocks, but they may be in a very unsatisfactory condition due to fracturing, alteration, or other causes. The practical problems of dams on the granitic rocks are concerned with faults, shear zones, weathering and disintegration, and zones of alteration.

Like all rocks granites may be cut by faults and shear zones, and they are usually intersected by rather regular systems of joints. The joints may be somewhat open near the surface and may provide possible paths of leakage, necessitating grouting. Since these rocks are insoluble, however, water passing through cracks cannot enlarge them and open channels cannot occur. The granitic rocks are all susceptible to weathering, and in general the coarse-grained rocks are less resistant than the fine-grained ones. Sometimes the feldspar in the granite kaolinizes and destroys the bond between the crystals,

with the result that an apparently sound ledge breaks down easily into gravel. Sometimes in a biotite granite the biotite hydrates and swells, causing the rock to disintegrate. In warm, humid regions, weathering may penetrate to great depths.

Many successful dams have been built upon granite and similar igneous rocks. The Grand Coulee Dam and Diablo Dam in Washington, the O'Shaughnessy Dam and Morris Dam in California, the Arrowrock Dam in Idaho, and the Buchanan Dam in Texas may be mentioned.

Rapide Blanc Dam.—The Rapide Blanc Dam on the St. Maurice River in Quebec, Canada, a concrete gravity dam 145 ft high, had an almost ideal site on gneissic granite; yet the first site examined, about half a mile upstream, was abandoned because of the badly fractured condition of the rock. Sheet jointing dipped toward the river in the right bank, and the natural tendency for blocks to move down the joint planes had been accentuated by action on a nearby fault. The present site was then investigated and it was found that the sheet jointing was not well developed, that joints did not dip toward the river, and that they had not been affected by faulting. Sound granite was at or near the surface everywhere, and a better dam site is difficult to find.

Forks Dam Site, San Gabriel Canyon.—The Forks Dam site in San Gabriel Canyon, California, illustrates the extreme changes produced by faulting, crushing, alteration, and decay in originally strong granitic rocks. The site is at the junction of two fault zones, and the rock is cut in every direction by innumerable faults and crush zones that divide it into blocks of varying size, which are themselves fractured. In addition to the crushed and fractured condition of the rock, much of it is deeply decayed and disintegrated and alteration extends to depth along the fault zones.[33]

Dams on Schist and Slate

Schist and slate are metamorphic rocks which have distinctive physical characteristics in regard to dam foundations. The principal characteristics of these rocks from the foundation viewpoint are foliation and cleavage. The schists have a foliation which may be wavy. They split easily with the foliation but break with difficulty across it. The cleavage of slate is smoother and more regular and may divide the rock into thin sheets.

The principal practical problems of dams on slate or schist are sliding and stability of abutments. Sliding might take place on either the cleavage planes of slate or the foliation planes of schist. Mica schist probably has the lowest resistance to sliding of any of the schists. Cleavage planes are usually not actual fractures but they are planes of weakness. Schists and slates are also intersected by joints but these are often not so well developed as in sandstones and limestones. Neither slate nor schist disintegrates as do the uncemented shales. Some schists are very resistant to weathering, whereas others weather easily and deeply.

[33] "Serious Defects in Foundation Rock Stop Work on San Gabriel Dam Project," *Engineering News-Record*, October 31, 1929, pp. 699–701; also "State Denies Permission to Build San Gabriel Dam as Proposed," *Engineering News-Record*, December 5, 1929, p. 895.

Schists vary greatly in strength and each variety must be tested. Some slates are moderately strong rocks as shown by compression tests on three slates which had a strength of 17,700 to 19,199 lb per sq in. transverse to the cleavage. The shearing resistance of both slate and schist is much greater across the cleavage than parallel to the cleavage. The relation of the cleavage to the dam, therefore, is very important.

Fifteen Mile Falls Dam.—A good example of a successful high dam on schist is the Fifteen Mile Falls Dam on the Connecticut River in northern New Hampshire. This is a concrete, gravity dam, 175 ft high, completed in 1930, resting upon schist which is in part slaty. A very thorough geological investigation was made and was continued during the construction period.[34]

Prettyboy Dam.—Some of the problems due to weathering of schist and to the difficulties of excavation in some schists were illustrated at the Prettyboy Dam. This is a gravity, concrete dam, built for the water supply of Baltimore, Md. In the lower part of the west abutment sound schist was about 8 ft below the surface, but in the upper and westernmost part of this abutment sound rock was more than 100 ft deeper than expected, which necessitated greatly increased excavation. The weathered rock had been dropped down behind the fresh rock, by a fault, giving a deceptive appearance to the face of the abutment. Even where the rock was sound, the ease with which shooting split the rock along the planes of schistosity made it difficult to obtain a good bottom for the foundation excavation and necessitated much additional excavation. This difficulty was finally obviated by the use of wire saws and the avoidance of explosives.[35]

St. Francis Dam.—In addition to the possibility of sliding on cleavage planes, there exists the danger of injury to the dam from instability of the abutments if the strike of the schist parallels the valley and the dip is parallel to, or flatter than, the valley wall. This was illustrated at the east abutment of the St. Francis Dam. The east wall of the canyon is formed of mica schist with the planes of schistosity dipping steeply to the northwest parallel to the wall of the canyon. Any undermining of the canyon wall would remove the support of masses of schist above, which would avalanche into the canyon. Most of those who investigated the failure of the dam concluded that the schist was undermined by the rush of water from the break in the western part of the dam and that large slides then occurred; but one commission concluded that slides in the schist were largely responsible for the failure of the dam.[29] In any case, such an unstable abutment is most undesirable, and such conditions are not rare since the slope of canyon walls is frequently determined by planes of schistosity or cleavage, or even by the bedding of sedimentary rocks.

Conclusions

From the preceding examples and discussions, the many difficulties that may arise from not meeting the geological conditions at a dam site are apparent.

---

[34] "Geology of Fifteen Mile Falls Development," by Irving B. Crosby, *Civil Engineering*, January, 1934, pp. 21–24.

[35] "Redesign and Construction of Prettyboy Dam," *Engineering News-Record*, July 20, 1933, pp. 63–67.

The many circumstances which may reduce the value of a dam, due to leakage or other defects, or may greatly increase the costs, unless properly provided for, have been discussed and it has been emphasized that these may be predicted in advance. When they have been foreseen, and when the engineer has received full information about the foundation conditions, he can meet them, howsoever bad, unless he finds it uneconomic to do so. The same system of investigation which has revealed unfavorable natural conditions can usually find an alternate, more favorable site. Dams may fail from many causes, due to many types of defects in the foundation rocks; but there is nothing mysterious about these failures. They are all due to definite geological conditions which can be discovered in advance if scientific knowledge and experience are given adequate opportunity.

CONCRETE CONTROL

By I. L. Tyler,[36] M. Am. Soc. C. E.

Synopsis

Concrete suitable for use in dam construction must have the following characteristics: Strength sufficient to carry the loads and provide a required factor of safety; weight, in the case of gravity dams, to provide safety against sliding and overturning; durability to insure against weathering and erosion; impermeability to prevent water percolation and solution of cementing material; and continuity in order that the structure may act according to assumptions of design. These essentials are controlled by various factors which may be grouped under two headings: First, materials from which the concrete is made, and second, methods of using the materials. Materials available and methods for using them scarcely can be considered separately since one is often dependent on the other, and the design of concrete therefore requires consideration of materials and construction procedure as well as type of structure in which it is to be used. It follows, then, that problems of selecting materials and determining construction procedures and schedules should be considered as problems of design.

Although knowledge concerning properties and behavior of concrete under various influences has been enormously increased during the last few years, an urgent need for additional investigations, mainly in the field of mass concrete, in which the greatest strides have recently been made, still exists. Application of knowledge gained from laboratory tests and from field observations has aided greatly in bringing the quality of dam structures to its present state. Future progress depends on utilizing available information to the fullest extent as well as on investigating further the many phases of concrete behavior still not well understood.

This paper is an attempt to describe, very briefly, the present state of progress in concrete manufacture and control as applied to construction of dams, with some mention of factors that may be of importance to future developments. As the scope of this paper will permit only a small amount of detailed discussion, a list of references is included in the Appendix for the benefit of those who may wish to study some of the subjects mentioned.

Materials

Cement.—In addition to having properties assuring strength, durability, and impermeability of concrete, cement for large dams should have a low rate

[36] Concrete and Materials Engr., Pennsylvania Turnpike Comm., Harrisburg, Pa.

of heat evolution during hardening and a low total heat liberation. Search for such a cement has led to the development of low-heat portland cement and portland-puzzolan cements of various types. Extensive investigations before and during construction of Boulder Dam were largely concentrated on heat of hydration of cement because it was recognized that temperature effects were largely responsible for cracking in large masses of concrete. Out of these investigations and further studies on behavior of several concrete dams came a realization of the importance of another property of concrete—that of its ability to withstand temperature strains without cracking. Recent tests indicate that success of the low-heat cement in reducing cracking may be due as much to this property as to its favorable characteristics of heat liberation.

Following the development of low-heat cement and its use in Morris Dam, in California, and Boulder Dam, on the Colorado River, a modified portland cement came into general use for dam construction. This cement has a more closely controlled chemical composition and physical properties than the normal portland cement, and its heat of hydration lies about midway between the normal and low-heat types. It has part of the advantages of the low-heat cement without its disadvantages, and for a time it appeared that the modified cement best filled the general requirements of dam construction. Modified portland cement has been used successfully in many large dams and is still considered by many to have distinct advantages for some types of dam construction.

At the present time (1940) the use of low-heat portland cement for large dams is rapidly increasing and as problems attending its use are overcome, low-heat cement may be expected to displace the other types except in locations where portland-puzzolan cement may be more economical (or in extremely cold climates where construction difficulties may make its use impractical). Usual specifications for the three types of portland cements are compared in Table 10.

Portland-puzzolan cements present individual problems and scarcely can be considered as a class at the present time because of the variety of puzzolanic materials that may be used. This type of cement is essentially a mixture of portland cement and some form of active silica. Present thought is in the direction of intergrinding portland cement clinker of about "modified" composition with suitable puzzolanic material to form an intimate mixture of greater uniformity than could be obtained by batching the materials ground separately and depending upon the less thorough mixing during the manufacture of concrete. Portland-puzzolan cement has been used successfully in Bonneville Dam, in Oregon and Washington, and undoubtedly more will be heard of it in the future.

No definite statement of relative durability of concretes containing these types of cement can be made. It is generally agreed that concrete containing low-heat cement which has not been properly protected and cured during its early hardening stages is inferior in resistance to weathering to concrete containing normal or modified portland cement of equal curing history. Durability tests made at early ages are generally unfavorable to low-heat cement because of its slow rate of hardening. Freezing and thawing durability tests

on properly cured concrete specimens at ages of approximately one year are not conclusive in establishing a definite superiority of any one type of cement, variations between brands of cement generally being greater than variations between types. Freezing and thawing tests have shown conclusively the necessity for proper protection and extended curing of low-heat cement concrete which is to be exposed to weathering.

TABLE 10.—COMPARATIVE SPECIFICATIONS FOR THREE TYPES OF PORTLAND CEMENTS

Item	Type of cement	Ignition loss[a]	COMPOUND COMPOSITION[b]		PHYSICAL PROPERTIES[c]		
			Tricalcium aluminate	Tricalcium silicate	Fineness	Strengths[h]	
						7-day	32-day
	(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	Normal portland	4.0	....	....	. ....[d]	275[f]	350[f]
2	Modified	3.0	8	35 to 55	1,600 to 2,200[e]	1,500[g]	2,500[g]
3	Low heat	3.0	7	• 35	1,700 to 2,300[e]	1,000[g]	2,000[g]

[a] *Chemical Analysis.*—Maximum percentages; other items of chemical analysis, common to all three items, are: Insoluble residue, 0.85% maximum; sulfuric anhydride, 2% maximum; magnesia, 5% maximum; and ratio of iron oxide to aluminum oxide, 1.5% maximum (except item 1).
[b] *Compound Composition.*—Maximum percentages; in addition to values in Cols. 3 and 4, the following applies to item 3 only: Dicalcium silicate, 40% to 65%; and tetracalcium alumina ferrite, 20%.
[c] *Physical Properties.*—Standard steam soundness tests on neat cement pats showed each item to be sound. The initial set (using a Gillmore needle) occurred in 1 hr (minimum), and the final set occurred in 10 hr (maximum). [d] Measured in terms of the percentage passing or retained on a 200-mesh screen. [e] Measured in square centimeters per gram (by Wagner turbidimeter). [f] Minimum tensile strength briquets. [g] Compressive strength of 2-in. mortar cubes. [h] In pounds per square inch.

Fineness to which a cement is ground has been shown to have a marked effect on physical properties of fresh concrete in which it is used. It also affects rate of hardening and heat liberation. Coarse cement has a rapid rate of settlement in water and when used in concrete permits the movement of mixing water vertically through the concrete, producing the phenomenon of bleeding. Surface effects, such as sand streaking and water and laitance accumulations at the tops of concrete lifts, are common results of bleeding. Internal effects of bleeding are not well understood. For reasons which are still somewhat in doubt, low-heat cement seems to have a greater tendency toward bleeding than normal or modified cements of equal fineness. Therefore, somewhat finer grinding of low-heat cement is desirable. Sufficiently accurate control of fineness has been made possible by the development of several devices for determining approximately the surface area of cements. Finely ground cements harden and produce heat more rapidly than coarser cements because of the greater area of cement exposed for hydration. For mass concrete the high rate of heat liberation is considered undesirable, and modified cements are usually limited to about 2,100 sq cm per g (by turbidimeter) and normal portland to a somewhat lower value for such use. Experience indicates that for concrete mixes of 1 bbl of cement or less per cu yd, fineness of low-heat cement could be at least 2,200 sq cm per g with advantages of decreased bleeding and increased workability outweighing the disadvantage of increased

heat of hydration. It is to be expected that bleeding and other characteristics of cement may be governed by its particle size distribution as well as by its specific surface.

Aggregate.—Ordinarily aggregate for concrete dam construction must come from the cheapest suitable source—in most cases the source nearest the dam site. As dams are usually built in outlying districts, it often happens that the available aggregate has no record in concrete upon which to base conclusions as to its suitability for the intended purpose. It then becomes necessary to determine by experimental means the various properties of the material which might affect its behavior when used in concrete. In the past the tendency has been to rely on visual inspection and the personal opinion and judgment of the engineer. In many cases much may be learned by careful observation of behavior of the material in outcroppings or exposed locations of the deposit, and evidence gained in this manner should in no way be minimized. However, present practice is to augment such information greatly, particularly in doubtful cases, with results obtained by subjecting the material to various durability tests, microscopic examinations, and chemical and other tests in order to furnish the engineer with more complete data on the material with which he has to deal. An unknown aggregate proposed for use in an important dam might be investigated by applying tests as follows: (1) Chemical analysis, (2) petrographic analysis, (3) absorption, (4) specific gravity, (5) abrasion, (6) sodium sulfate or magnesium sulfate test for soundness, (7) strength of the aggregate in compression, (8) strength of mortar and concrete containing the aggregate, (9) freezing and thawing tests on aggregate and on concrete containing the aggregate, (10) expansion due to temperature change, (11) expansion due to moisture change, and (12) thermal properties. Satisfactory performance in these tests and no unsatisfactory indications in the field investigations would practically assure that an aggregate would give good service in a concrete structure. Conflicting indications would be cause for caution or further investigation, if not rejection, of the aggregate.

Concrete aggregates may be of two general types—natural or manufactured. Suitable natural aggregates, usually siliceous in composition, are found in river or glacial deposits at some distance from their point of origin. Weathering and erosion have eliminated unstable material, leaving sound particles with rounded corners which are well suited to use as concrete aggregates. When such a deposit of reasonable particle size distribution is available, the problem of aggregate supply is solved. In areas where deposits may contain appreciable quantities of unsound chert, shale, soft sandstone, or other undesirable material, the problem becomes involved and the manufacturing of aggregates from native rock may have to be considered.

Manufactured aggregate crushed from suitable deposits of various types of rock has been used successfully for some time as coarse aggregate for concrete. Use of crushed sand has not always been entirely satisfactory, leading in many localities to rejection of manufactured sand in favor of natural sand, even at a considerable premium in cost. The unsatisfactory performance of crushed sand in concrete has undoubtedly been due to unsound rock in some cases, but probably has been due more often to poor particle shape and poor particle

size distribution and consequent high water requirement of the concrete in which it has been used. Both coarse and fine crushed aggregates are being used at the present time in producing concrete which is reasonably certain to compare favorably with concrete containing the best of natural aggregates. Fig. 7 shows the effect of methods of manufacture on the particle shape of aggregate. The aggregate shown in Figs. 7(*a*) and 7(*b*) requires appreciably less cement than that shown in Figs. 7(*c*) and 7(*d*).

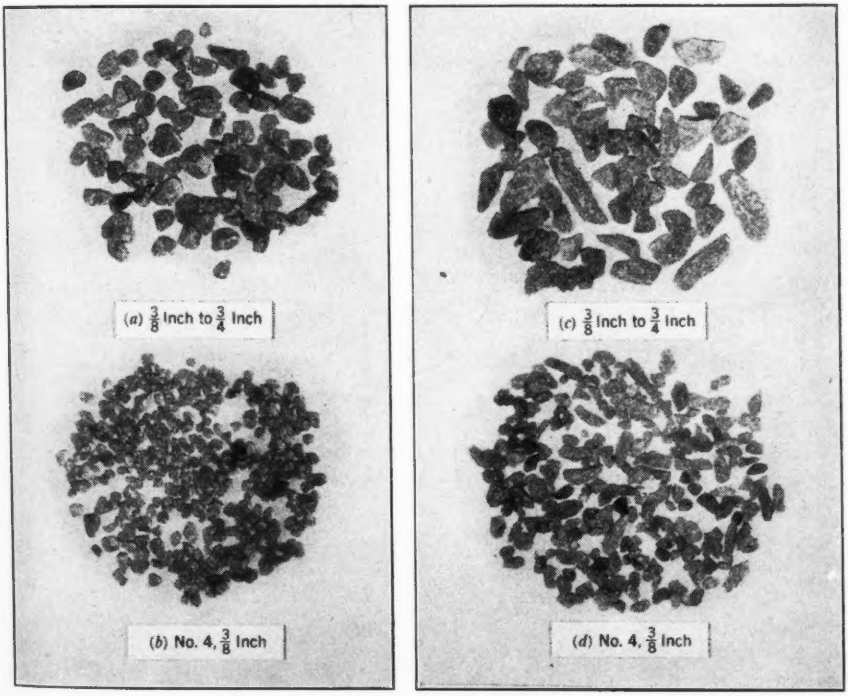


FIG. 7.—PARTICLE SHAPES PRODUCED BY DIFFERENT MANUFACTURING PROCESSES

Regardless of which type of aggregate is used, it is essential that processing plants and handling methods and equipment be such that the combined aggregate at the mixing plant will be clean and of a uniform grading which may be controlled within limits appreciably affecting the quality of concrete. In order to permit accurate control of mixing water, aggregates must be of uniform moisture content when delivered to the mixing plant. Usual best practice is to wash the coarse aggregate, either before or after screening to size, and to wash and classify sand by means of wet classifiers. For mass structures in which aggregate sizes of 6 in. or larger are used, four sizes of coarse aggregate are usually required and either one or two sizes of sand in order to satisfy minimum permissible variations in combined aggregate grading. Aggregates for mass concrete are usually separated into the following nominal size groups:

Size	Group
3 in. to 6 in.	Cobbles
$1\frac{1}{2}$ in. to 3 in.	Coarse
$\frac{5}{8}$ or $\frac{3}{4}$ in. to $1\frac{1}{2}$ in.	Medium
No. 4-mesh to $\frac{5}{8}$ or $\frac{3}{4}$ in.	Fine
Smaller than No. 4-mesh	Sand

Accurate sizing of the different aggregates, control of breakage (usually most severe in the larger sizes), and elimination of segregation during stock-piling, reclaiming, and transporting are essential. Segregation is present to some extent in almost every handling operation and in some, such as conical stock-piling, may become very serious. It is largely because of segregation that accurate screen sizing and control of breakage assume such importance, since segregation may cause high concentrations of unwanted particle sizes for short periods during delivery of material to the mixers, even if the total percentage of the unwanted material may be comparatively small during a day's operation. Methods of crushing, screening, and handling aggregates are seldom duplicated, and each new project has its problems which must be met. Careful consideration of these problems in the design of the aggregate plant is usually much more effective than any attempts at correction after the plant is in operation.

Ample provision for drainage of aggregate to constant moisture content is necessary in order that control of the water-cement ratio of the concrete may be effective. The time required for aggregates to drain properly varies with the size of the stock pile and the size and grading of the material. Clean, coarse aggregate, even in large stock piles, will normally drain sufficiently in a few hours, whereas large piles of sand may require several days to drain to a reasonably constant moisture content.

Mixing Water.—Mixing water for concrete seldom receives much attention because few instances have been recorded that indicate failure of concrete because of unsatisfactory mixing water. Exceptions to this have been cases in which water containing industrial wastes has been used, and the concrete in these cases has shown disintegration directly traceable to impurities in the water. Concentrations of silt and organic matter in most streams or rivers of low industrial waste pollution are usually insufficient to affect properties of concrete noticeably. Excessive quantities of silt should be removed by clarifiers and organic material by sand filters or other means. Water containing chlorides, sulfates, or other soluble materials in unusual amounts should be viewed with suspicion. Mixing water for any important structure should be tested for impurities of harmful nature which approach dangerous limits. Tests should include use of the water in mortar or concrete test specimens.

Concrete Control

Proportioning.—In the absence of actual test data the water-cement ratio offers the most acceptable measure of potential concrete quality. This is true with possible rare exceptions, because, with proper curing and attention to the other factors of concrete quality, the water-cement ratio is an accurate measure

of the quality of the weakest, least durable portion of portland cement concrete —the cement paste. The presence of some water is necessary to the hardening process of the cement paste (formation of cement gel), and the presence of a much greater quantity of water is necessary in order that the concrete may be consolidated in place with a minimum of void spaces between particles of aggregate and cement. The fact that use of more cement in a concrete mix raises the quality of the cement paste (water content of concrete at fixed workability changes little with cement content), and at the same time adds to the quantity of the least stable ingredient, presents an involved problem for which there is not a definite answer at present.

Designing of concrete mixtures involves the combination of available materials in such a way as to produce concrete of predetermined quality most economically. Normally this resolves itself into determining the combination, or possibly combinations, of aggregates of different particle size which will require the least amount of cement paste of fixed water ratio to give plastic, workable concrete under actual field conditions of mixing, transporting, and placing. The first step in laboratory investigation of concrete mixes, therefore, is in the direction of aggregate grading. It is seldom, if ever, possible to obtain the aggregate grading at the mixing plant which the laboratory tests indicate to be the most desirable, and it is not often that laboratory gradings are entirely successful when used in large-scale concrete production because of segregation during mixing and handling and other deficiencies in plant or handling methods. In spite of such limitations, intelligent use of laboratory information, suitably modified to approximate field conditions, is of inestimable value in predicting material requirements and quality of concrete.

Many methods have been proposed for determining theoretical aggregate gradings, each with the idea of using the minimum amount of cement for a given water-cement ratio. Most of these methods approximate a condition such that quantities of each particle size group bear a fixed volumetric relation to the next larger (or smaller) size group. Probably the best known and most used grading formula is the familiar

$$p = \left(\frac{d}{D}\right)^n \qquad (4)$$

in which p is the proportion passing a given screen size d; D is the maximum size of aggregate; and n is a factor roughly measuring harshness of the mix. Plotting the percentage passing against screen size on double logarithmic paper makes a very convenient method of handling this equation. The slope of the grading curve (then a straight line) is measured by n. The use of theoretical gradings must be accompanied by judgment on the part of the engineer. It is seldom if ever possible to calculate an aggregate grading and proportion a concrete mix without experimental data or actual trial batches of concrete as a guide. Too close adherence to theoretical gradings usually leads to undersanded and harsh concrete mixtures.

Batching and Mixing.—Concrete batching and mixing equipment has developed rapidly during the last few years. Development has been largely

in the direction of increased capacity because of demands of the many large construction projects. Batching equipment has improved notably in accuracy and precision of operation, as well as in handling capacity; but concrete mixers, with the exception of those on a few well-known projects, have not received the attention necessary to force the much needed development in this class of construction equipment.

Automatic weigh batching equipment is rapidly displacing manually operated batchers on large concrete projects because of its greater uniformity and precision in operation and speed in batching. Manual batching may be of equal or even greater precision over short periods of carefully supervised operation; but fatigue of the operator and pressure for high rates of concrete production often cause erratic batching. Weigh batching of water is generally preferred to volumetric batching, particularly in plants where automatic recording of quantities of materials is required. The better class of batching equipment has been developed to a point where it is superior to most other important items of plant equipment and operating procedures that affect quality of concrete.

Charging of mixers and the mixing operation are beginning to receive some careful attention. The two are considered together because investigations have shown that charging sequence of ingredients of the concrete batch may materially affect the quality of concrete produced in some mixers during the normal mixing times required. Heretofore about the only attention paid to charging sequence has been with the idea in mind of preventing building up of materials behind the mixer blades. It has been the practice for construction organizations to aid manufacturers in developing other items of construction equipment most essential to concrete production, handling, and placing; but until recently the development of concrete mixers has rested almost entirely with the manufacturers. Since it would scarcely be practicable for a manufacturer to do much experimenting with a large mixer, except in collaboration with some going construction project, the future progress in mixer design will depend largely upon such cooperation. An appreciable amount of significant experimental and development work has been conducted with marked improvement in mixer performance. Further progress may be expected in the future since serious attention has been directed to the problem.

The most important factor governing quality of concrete is control of mixing water. Modern batching equipment provides means for adding water in accurately determined amounts. This often gives a false sense of security to engineers who do not fully appreciate the difficulties involved in determining the quantities of free water carried by aggregates. Including sand, aggregates often contain more than one third of the total amount of mixing water required for the manufacture of concrete. So long as these moisture contents remain constant and are determined accurately, precise control of concrete mixing water is possible and full advantage of good batching equipment may be realized; but if the moisture contents vary widely, the value of excellence of batching equipment is greatly reduced. Development of consistency meters as indicators of mixing-water content offers promise where constant aggregate gradings are known to exist.

Transportation of Concrete.—Concrete should be transported quickly from mixer discharge to the point of deposit, and without segregation of materials. Segregation occurs in every handling operation to some extent, including discharging of mixers and dumping of concrete buckets; and it is important, therefore, that the number of additional handling operations be reduced to a minimum. With large aggregate sizes, best results have been obtained by moving the concrete, preferably batch by batch, from mixers to form with no storage between, thus assuring that each batch reaches the forms with the materials it should contain. Bottom dump concrete buckets, loaded directly from the mixers or from intermediate transfer devices handling single batches or multiples of batches not exceeding the bucket capacity, are necessary in this procedure. Concrete transfer systems using methods of moving concrete in thin streams with transfer points and storage hoppers are generally considered undesirable from a concrete quality standpoint because of the comparatively large degree of segregation that may take place and the loss in flexibility of plant operation. Such systems are used occasionally for economic reasons and may be justified in some instances; in general they should be avoided. For concrete containing smaller maximum-size aggregate, pumping offers a feasible method of transportation under suitable conditions.

Concrete Placing.—Modern construction practice requires that concrete shall be deposited approximately in its final position in the forms. For dam construction, bottom dump buckets handled by cranes or cableways have come into general use as best fitting the usual needs. Buckets of 3-cu-yd, or less, capacity are usually dumped manually, whereas larger buckets are often dumped by power in order to decrease danger to workmen and to save time. Both square and round buckets are used, square buckets generally with comparatively fast uncontrolled discharge for mass concrete, and round buckets mostly of the controllable discharge type for either mass concrete or for concrete to be placed in thin or reinforced sections.

In starting a lift of concrete in a dam, normal procedure is first to deposit a batch of sand-cement mortar which is wire-broomed thoroughly into the old concrete surface or rock foundation, leaving a layer perhaps ½ in. thick. Some question still remains concerning the proper stage of dampness of the old concrete surface at the time it is to receive the mortar. It is generally agreed that the old surface should not have been allowed to dry out before starting the following lift of concrete, but there is evidence that the surface should not be wet at the time mortar is deposited. Presence of even small pools of water over the surface is highly undesirable. Some authorities believe that the best bond is obtained when the surface has been allowed to dry to a natural-surface, dry condition just before concrete placement is started. To serve its intended purpose most effectively mortar must be of a consistency such that it may be brushed readily into small depressions of the old surface. The water ratio of the mortar may be that of the concrete that is to follow, or slightly lower.

Placement of concrete should follow immediately after spreading the mortar, covering the grouted area as soon as possible. In the placing of mass concrete, it is general practice to deposit the contents of each concrete bucket in adjacent piles, vibrating each pile by means of internal vibrators to a thickness which is

readily penetrated to full depth by the vibrator and joining each pile in such a manner that individual deposits cannot be distinguished after the operation is completed. When small buckets of comparatively soft concrete are used, no particular difficulties are encountered; when the contents of buckets containing 6 to 8 cu yd of relatively dry concrete are being deposited, something of a problem is presented, requiring excellent vibrating equipment and close supervision in order to assure uniformity and complete compaction of the mass. Present tendencies are strongly in the direction of drier concrete handled in large buckets because of lowered placing costs. This is particularly true on dams using cableways for concrete placement. There is considerable question in the minds of many engineers about the desirability of the large buckets when viewed from the standpoint of concrete quality.

FIG. 8.—"NO SLUMP" CONCRETE BEING PLACED BY VIBRATION

There is little doubt that present tendencies toward drier concrete are generally justifiable on grounds either of quality or of costs. Attempts to use very dry concrete mixes are likely to be unsuccessful, however, unless highly efficient, well-serviced vibrating equipment is available and unless the difficulty of handling the dry concrete is thoroughly appreciated by those doing the work. Opinions concerning the advantages of different types and makes of vibrators vary greatly among construction men and engineers, but there is agreement on the value of internal vibration of concrete as opposed to surface vibration (see Fig. 8). For mass concrete placement, the choice of vibrators lies between electrically driven and compressed-air machines. When used in dry mass concrete, most vibrators require two men for continuous operation. Electrically driven vibrators operating at approximately synchronous speeds offer dependability to a high degree, but they are heavy and until recently could not be used at high vibrating speeds. Air driven vibrators generally

have advantages of high speeds and light weight, but are dependent on a uniform air supply for satisfactory operation. Much controversy has taken place over high-amplitude, low-speed operation of vibrators versus low-amplitude, high-speed operation. At this time there is a definite lack of information on the subject, but it appears likely that for each type of concrete mix there is some combination of frequency and amplitude best suited to placement of that particular mix. In general, vibrating frequencies are being increased with beneficial results, although recent discussions on vibration of concrete, emphasizing the advantages of high frequencies, have led to a general disregard of the necessity for sufficient amplitude. Size and shape of the vibrating element also have marked effects on vibrator performance.

At considerable risk of starting a controversy, it seems appropriate to consider the problem of placing concrete in wet weather. This subject is neatly avoided in nearly all specifications for dam construction, although it is known by all careful observers of concrete that it is not possible to place the same quality of concrete in even a light rain that can be placed under favorable dry-weather conditions. The most serious danger exists at the beginning of a concrete lift where mortar covering the old concrete may be diluted with rain water to such an extent that it is worthless for its intended purpose. The result may be a porous joint. In order to forestall the ready argument that "you would never get a dam built if you prohibited concrete placement in the rain," it may be noted that rolled earth dams are often built in damp climates and the nature of equipment used in their construction definitely limits the work to dry weather. Admitting the validity of the argument quoted does not alter the fact that serious damage may and sometimes does result from the common practice of paying too little attention to the effects of rain on the placement of concrete.

Cleanup, Curing, and Finishing.—Preparation of concrete surfaces upon which additional concrete is to be placed is considered to be a part of the general problem of "Construction Joints," which subject is presented by Mr. Steele in this Symposium. It is sufficient to note that construction joints are probably the weakest locations in any concrete dam, and consequently their treatment deserves very careful consideration.

The effects of concrete quality and placement methods on effectiveness of construction joints are fundamental. In addition to causing surface laitance accumulations, concrete that bleeds appreciably may seriously affect the upper layer of the concrete itself if the surface is disturbed. Agitation causes remixing of surface water with concrete near the surface and may result in a concrete of higher water ratio than that originally placed. Evidence of this has been noted in cores drilled across construction joints which, in many cases, break below, rather than at or above, the joint between lifts. A well-compacted surface with aggregate particles embedded in the concrete is essential to a good construction joint. In general, surface vibration should be avoided because of the danger of bringing mortar to the surface and remixing it with accumulated water. Traffic of placing crews and others over completed lifts should be prohibited until the concrete has hardened.

Curing of concrete includes its protection from extreme temperatures, drying out, and other possible damaging agents after the concrete has been placed in the forms. Protection must be afforded until the concrete has developed sufficient strength to withstand the effects of conditions under which it is to function. Protection from excessively high temperatures and drying out may usually be accomplished to best advantage by continuous sprinkling of surfaces. Forms left in place help greatly. With normal portland or modified cement the sprinkling should be continued for at least fourteen days. With low-heat cement, the time should be extended to at least three weeks and preferably longer. Protection from drying out alone may be afforded by coating concrete surfaces soon after removal of forms with certain sealing materials which retard moisture loss from the surfaces. Care should be exercised in selecting these materials, however, as only a few have been proved to be effective. Black coatings, when exposed to the sun, should be painted with some heat-reflecting paint to prevent excessive temperatures at concrete surfaces. Thick concrete sections are usually affected only slightly by drying out, except at the surface. Consequently, sprinkling of mass concrete surfaces has little to do with moisture-volume change or cracking inside the mass, except as surface cracks may furnish weaknesses and provide starting points for temperature cracking. Thin concrete sections are much more affected by surface-moisture loss because, in addition to affecting strength and durability of the surface, drying may remove enough moisture from the interior of the structure to produce large shrinkage and resulting cracking.

Cold-weather protection, often a problem with concrete containing normal or modified portland cement, presents considerably more difficulty when low-heat cement is used. The rate of hardening of any concrete is very slow at temperatures of 40° F or lower, and for placing temperatures approaching this range, a considerably longer time of protection against freezing must be provided than for placing temperatures 10° or 15° F higher. As it is undesirable to raise mass concrete temperatures above a minimum which is absolutely necessary (probably between 40° and 50° F), provision for protection against freezing for two weeks after placing may be necessary in some cases.

The essential requirements in obtaining a concrete finish surface that will withstand erosion and weathering are low water ratio and proper compaction. High cement content, undesirable from many standpoints, is usually necessary in order to provide the required quality of cement paste. In monolithic finishes on concrete lifts of appreciable depth, it is not possible to obtain a lasting surface if bleeding in the concrete takes place because of water gain in the top layer while working the surface with trowel or float. In all cases the minimum amount of work on the surface needed to produce the desired texture or appearance will also produce the most lasting surface. Concrete surfaces that can be made by placing concrete against forms are nearly always more superior in resistance to erosion and weathering than those finished by floating or troweling.

Cracking.—The dangers of cracking in a concrete dam may be very serious, affecting structural stability and the lasting qualities of the structure. Effects

on stability are possibly less than might be expected because of tendencies to heal and reunite disconnected sections if openings are small and if water leakage or frequent movement does not interfere. Effects on durability are probably greater than might be expected because they are not immediately apparent. Cracks provide passages for percolating water and encourage the solution of cementing materials in the concrete. Presence of water in cracks permits the full disruptive forces of repeated freezing to accelerate the normal surface disintegration greatly.

Cracking in concrete may be due to temperature effects, moisture volume changes, foundation deflections, load deformations, or a combination of these. In thick concrete dams it seems conservative to estimate that more than 75% of all cracking is due to temperature effects introduced during the construction period. In multiple-arch and other hollow dams the effect of moisture volume changes in thin arch sections or slabs may be the largest factor contributing to cracking. Cracking in buttresses of moderate thickness in these types of dam may be caused by a combination of moisture and temperature changes. Cracking of concrete produced by foundation deflections or load deformations which are independent of temperature or moisture volume change effects present problems outside the scope of this paper.

Temperature stresses that produce cracking in thick concrete sections might be considered as being of two classes: Those produced by temperature changes in a block of concrete acting as an unrestrained unit, and those caused by restraint of one section by another. Advantages of attempting to distinguish between the two are of value for purposes of discussion only since basically all cracking is due to restraint of one kind or another.

Cracking due to temperature changes in an unrestrained block of concrete usually takes place early in its curing history, often within a few weeks after the block has been cast and sometimes within a few hours. In most cases such cracking takes place while the interior of the block is still rising in temperature, although sudden cold weather may produce the same effect at much later ages when internal temperatures are falling. It is due to inability of the cooling layers near exposed faces to elongate with the inner layers which may be increasing in volume due to temperature rise. Tendency to produce such cracking is governed by properties of the concrete, changes in temperature, and the time through which temperature changes take place. Insulating effect of forms, effect of curing water, and concrete placing temperatures are partly controllable factors that may influence such cracking. The most serious uncontrollable factor is varying air temperature. It has been observed that concrete placed early in the spring season of the year is much less subject to cracking than that placed in summer or early fall because of generally favorable temperature conditions during seasons of rising temperature and unfavorable conditions during seasons of falling temperature.

Cracking due to restraint such as may be produced in concrete placed on rock foundations or in concrete that has cooled to nearly mean annual temperature is caused by tensile stresses set up during cooling of concrete above the joint. From a structural point of view, cracks produced in this manner are

more serious than those previously discussed because they are usually vertical and parallel to the axis of the dam, approximately in planes of maximum shear if they occur near the toe of the dam. Little is known about the development of these cracks because there is seldom opportunity for observation of their behavior except in hollow-dam buttresses or concrete walls where moisture volume changes may also be influential. It is likely that cracks started initially by temperature changes between interior and exterior are continued by a restraining action of the foundation during cooling of the concrete mass, resulting in cracks of zero opening at foundation and appreciable opening some distance above. In some instances such cracks have been known to extend upward to the downstream face of the dam.

Cracking of concrete in thin sections due to moisture volume changes is difficult, if not impossible, to eliminate entirely, but may be minimized by careful design of the structure and excellence of concrete quality. Proper spacing of artificial contraction joints, where structural requirements permit, will aid materially in controlling such cracking. Using highly impermeable concrete of high tensile strength will also aid greatly by slowing the rate of moisture change and by providing concrete more resistant to whatever tensile stresses are produced. Protection against moisture loss from concrete during early ages, by applying moisture-proofing membranes to exposed surfaces, has been at least partly successful; but so far as is known, little has been accomplished toward extending their effectiveness over long periods. Moist curing or its equivalent is essential during early ages.

Properties of materials which affect temperature cracking in concrete include the following: Rate of heat liberation and total amount of heat released by the cement during hardening, coefficient of thermal expansion of concrete, elastic modulus of concrete, tensile strength of concrete, plastic flow of concrete, and thermal properties of concrete. These items, together with foundation characteristics, construction scheduling and procedures, and weather variations, govern the behavior of a concrete structure in regard to temperature cracking. Efforts to eliminate the cracking by taking advantage of controllable factors affecting temperature changes have not been entirely successful to date, but results of such efforts have been most encouraging. Of the procedures most effective in producing conditions favorable to decreased temperature cracking, the following are considered to be of particular importance:

1. Use of low-heat cement or its equivalent in the lowest amount consistent with strength and other requirements (this usually means providing one concrete mix for the interior and another for exposed faces of the structure);
2. Use of low concrete-placing temperature by cooling mixing water or other ingredients when the placing temperature exceeds perhaps 50° F;
3. Elimination of extended exposures of long and high contraction joint faces;
4. Providing especial consideration to irregular foundations;
5. Limiting concrete lifts to a 5-ft thickness, or less, and reducing this appreciably when covering foundation or concrete which has been allowed to cool;

6. Scheduling of concrete placing so that placing rates are uniform, with sufficient time between lifts to permit dissipating an appreciable amount of the heat liberated by hardening of the cement; and
7. Artificially cooling the concrete in place.

For extremely large dams, artificial cooling is almost necessary if groutable contraction joint openings are to be obtained within a reasonable length of time after completion. Therefore, control of cracking may be accomplished best by taking full advantage of the artificial cooling, with as many of the other precautions as may be considered necessary. The effectiveness of artificial cooling on any concrete of mass proportions is well established, but to take full advantage of its effects the process should be started as soon after the placing of concrete as conditions permit—within a day or two if possible. For small dams, or dams of moderate size, a proper combination of the foregoing procedures can be made to reduce temperature cracking to a small amount, if not to eliminate it entirely, even without the use of artificial cooling.

Record Tests

Attempts to investigate old concrete structures on which no records of construction methods or materials tests are available furnish convincing evidence that future improvement in dam construction will be due in no small way to information gained from test data and construction records kept on present projects. Advanced practice in dam building is to investigate materials and concrete thoroughly, well in advance of construction, and to continue with record testing as the dam is built, thus leaving valuable information for future study in connection with the behavior of the structure. Preliminary investigations are often of an involved nature, including tests not well adapted to continuation as routine record testing. Compressive strength is still considered to be the best single index to concrete quality, provided relations between compressive strength and other qualities of the concrete exist and may be reasonably well established, even if the actual meaning of a compression test remains obscure. It is usual, therefore, to make compressive strength one of the more important record tests on concrete and to establish relations between strength and other properties whenever possible. Other common record tests include acceptance or equivalent tests on cement and aggregates and such investigations as may be found necessary by unforeseen effects of plant or methods on quality of concrete. In preliminary studies the various properties of concrete that may affect the behavior and lasting qualities of the completed structure are included. Usually considered, if not investigated, are properties as follows: Durability; workability and bleeding of fresh concrete; cement requirement; unit weight; compressive strength; elastic and plastic properties, Poisson's ratio; permeability; volume change due to temperature and moisture changes; thermal properties; and temperature conditions in masses.

Undoubtedly, increased knowledge of the properties of concrete as determined by laboratory tests has been responsible for greatly improving the

quality of dams in general, but recent investigations on dams during construction and subsequent loading have shown that the complete story of the structural properties of concrete can be learned only from studies of the full-size structure. Such investigations may well be considered a part of the record testing for an individual dam because they show whether or not the material behaves according to the intentions of the designers. They also serve the even more important purpose of furnishing general information on what actually takes place inside of a dam subjected to temperature changes and water load. Data from such investigations cover temperature, strain, deflection, joint openings, cracking, uplift, leakage, and other phases of dam performance. A considerable amount of such work has been in progress for some time and interesting and significant results have been obtained. Future investigations, particularly in the direction of stress-strain analysis and studies of temperature cracking in mass concrete, may be expected. It is at least an interesting fact that a 265-ft gravity dam has been known to deflect upstream $\frac{3}{8}$ in., against a rise in lake level of 150 ft, due to seasonal temperature effects.

Conclusion

The art of building concrete dams is becoming a science. Realization by engineers that concrete is a material that changes with variable quantities of its ingredients, and with manufacturing, handling, placing, and curing procedures, has done much toward attaining this objective. Efforts toward gaining control of the variables known to exist in concreting materials and procedures, and of first importance in producing structures of uniformly high quality, are becoming more successful, particularly on the larger dams. Investigations leading to a better understanding of the behavior of concrete under conditions of weathering, water pressure, loads, temperature and moisture changes, and other variables, are furnishing designers of dams with much needed information for future progress. There is still obvious need for further improvement in practically all phases of dam design and dam construction, but advances made in the past few years indicate that future progress may be even more rapid.

APPENDIX

References

(1) **"Special Cements for Mass Concrete,"** by J. L. Savage, M. Am. Soc. C. E., U. S. Reclamation Service, Denver, 1936.

(2) *Concrete Manual*, Bureau of Reclamation.

(3) **"Field Survey of Mass Concrete,"** by F. R. McMillan, M. Am. Soc. C. E., *Journal*, Am. Concrete Inst., May–June, 1938.

(4) **"Some Time Temperature Effects in Mass Concrete,"** by J. W. Kelly, *Journal*, Am. Concrete Inst., May–June, 1938.

(5) **"Cement Investigations for Boulder Dam,"** by R. E. Davis, M. Am. Soc. C. E., Roy W. Carlson, Assoc. M. Am. Soc. C. E., C. E. Troxell, and J. W. Kelly, *Journal*, Am. Concrete Inst., June, 1933.

(6) **"A Simple Method for the Computation of Temperatures in Concrete Structures,"** by Roy W. Carlson, *Journal*, Am. Concrete Inst., November–December, 1937.

(7) **"Calculations of Temperature Distribution in a Succession of Lifts Due to Release of Chemical Heat,"** by Robert E. Glover, November–December, 1937.

(8) **"Measured and Computed Temperatures of Concrete at Norris Dam,"** by Douglas McHenry, *Journal*, Am. Concrete Inst., November–December, 1937.

(9) **"Cracking in Mass Concrete,"** by R. F. Blanks, H. S. Meissner, Assoc. M. Am. Soc. C. E., and C. Rawhouser, Assoc. M. Am. Soc. C. E., *Journal*, Am. Concrete Inst., March–April, 1938.

(10) **"Temperatures and Stresses in Mass Concrete,"** by Roy W. Carlson, *Journal*, Am. Concrete Inst., March–April, 1938.

(11) **"Drying Shrinkage of Concrete as Affected by Many Factors,"** by Roy W. Carlson, A. S. T. M., 1938.

CONSTRUCTION JOINTS

By Byram W. Steele,[37] Assoc. M. Am. Soc. C. E.

Synopsis

The construction joint in concrete structures has taken on a new and added importance in recent years because of the tendency toward increased cracking caused by modern speed of construction and other factors. There is also gradually developing in the construction world a desire to substitute for the ragged haphazard crack, which is so objectionable from the standpoint of safety and appearance, a designed crack which will be architecturally acceptable and which will not develop undesirable features at a later date. Thus it appears that the construction joint, in its every phase, even if it is a necessary evil, should receive more careful consideration in the future than in the past. Where labor is cheap and materials are costly, this matter will undoubtedly receive more consideration than where labor is high-priced and materials relatively inexpensive.

A construction joint as treated in this paper is a formed or unformed, horizontal, vertical or inclined surface between masses of concrete placed at different times. The spacing, form, and method of construction of such joints are the leading topics considered herein.

On the question of the proper spacing of construction joints in general, and of vertical transverse joints in arch and gravity dams in particular, there is universal concurrence that they have been placed too far apart in the past and should be spaced closer together in the future unless other means are used to guarantee the elimination of cracking. It is also generally conceded that the spacing of joints for each dam should be decided upon only after due consideration of the type and height of the dam, profile of the dam site, foundation materials and conditions, speed of construction, climate, architectural treatment, whether artificial cooling of the concrete or aggregates is to be resorted to, the heat flow characteristics of the aggregates themselves, water-cement ratio of the concrete, height of the lifts, time interval elapsing between lifts, and last, and most important, the type of cement. There does not appear, however, in all the available literature on this subject, a set of rules for the spacing of joints that seems to be acceptable for general application.

It is interesting to note that 2,000 years ago the Romans were having much the same trouble with hydraulic structures that modern engineers are experiencing today and that the human tendencies then were just about on the same level

[37] Head Engr., Office, Chf. of Engrs., War Dept., Washington, D. C.

as now. In A. D. 97, Sextus Julius Frontinus, water commissioner of the City of Rome, wrote as follows:[38]

> "The heat of the sun is no less destructive to masonry than is too violent frost. Nor is greater care required upon any works than upon such as are to withstand the action of water; for this reason, all parts of the work need be done exactly according to the rules of the art, which all the workmen know, but few observe."

The data upon which the conclusions of this paper are based are assembled for reference in Table 11 and in the supporting comments in the Appendix.

Joint Spacing

It seems to be quite generally recognized that in dividing a dam into monoliths or blocks, the ideal spacing of the vertical construction joints normal to the axis of the dam would be a dimension that is varied from block to block to suit the profile of the dam site and the type and height of the dam. This ideal spacing is possible, but, due to serious objections from the architectural and construction standpoints, is rarely ever adopted. Economic considerations dictate that forms must be paneled and re-used until worn out, thus limiting the use of an ideal spacing, especially if variable.

It is believed, however, that a more rational approach to this subject than is generally made could be accomplished in connection with the construction of any dam if a construction-joint-spacing study was conducted early in the design period, before such features as outlet works, spillway gates, and a bridge over the dam begin to take form. Thus it would be possible to have sufficient data on hand to permit adequate consideration of the spacing of joints at the same time that bridge spans and spillway and outlet features are being considered. Regardless of the latter, however, a dam should be made structurally sound before being made architecturally beautiful.

A few years ago, 150 ft was not an unheard of spacing for construction joints in dams, and 75-ft to 100-ft spacing was common practice. At present, 15 m ($49\frac{1}{4}$ ft) in foreign countries and 50 ft in the United States seem to be the most popular spacings for the vertical joints normal to the axis of the dam, with the general requirement that each joint shall extend entirely through the structure. A notable exception to this generalization is the case of the Cignana Dam[39] in Italy, completed in 1928, in which there are three distinct systems of joints, as shown in Fig. 9. The principal ones are 30 m (98.43 ft) apart, cutting the entire section of the dam; the secondary ones intersect the entire width of the dam but extend only 19 m (62.34 ft) below the crest; and finally, a partial secondary system of joints midway between the principal and secondary extend only 19 m below the crest and from the upstream face to a drainage well, which is a short distance from the upstream face. Cignana Dam was the object of a series of observations to demonstrate the physical and mechanical behavior of the structure during and after the setting of the concrete, especially the joint openings for

[38] Excerpt from "The Water Supply of the City of Rome," by Sextus Julius Frontinus.

[39] "Longitudinal Contractions and Expansions Measured in a Large Concrete Dam," by Felice Contessini, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 161.

the different types of joints and at different seasons of the year, and in this respect is one of the most interesting demonstrations of record.

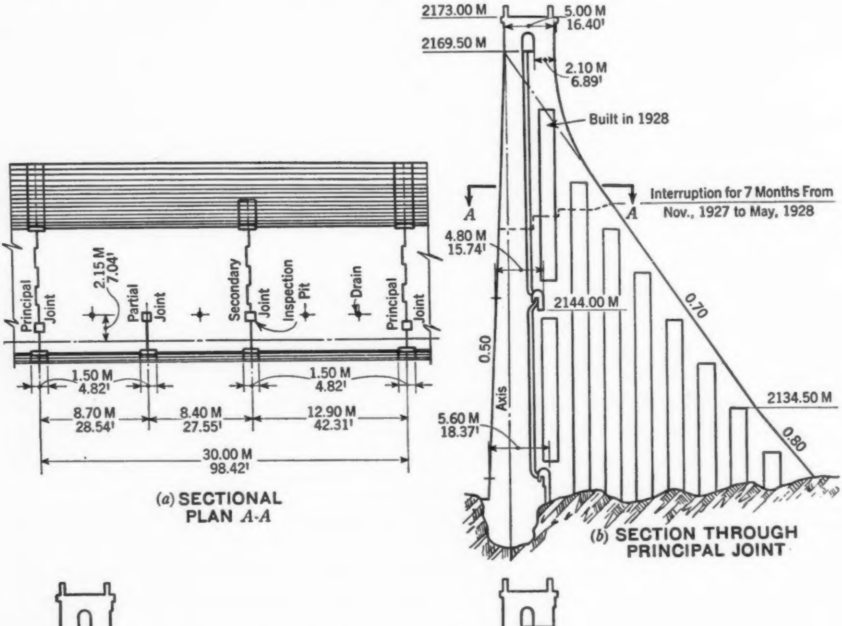


FIG. 9.—CONSTRUCTION-JOINT LAYOUT—CIGNANA DAM

There are numerous examples in dams, both in the United States and in other countries, of the lack of adequate consideration of the location of the vertical joints between monoliths on the canyon walls. It is common knowledge

that cracks invariably occur normal to the canyon wall slope rather than vertical, and yet engineers insist on the widest spacing that can be justified, and on carrying the vertical joints to rock in the most unnatural locations, if cracking is to be avoided—all for the sake of uniformity! True, it would not be practicable to form inclined joints so as to be normal to the canyon walls, but it is possible to locate and space vertical joints so that the tendency for inclined cracks to form is a minimum.

Perhaps this is a good place to admit that large monoliths can be built without objectionable cracking, but there are still so many "ifs" in the road to stumble over that to advocate returning to wide-joint spacing does not seem to be a desirable attitude to assume except for the dam where the necessary niceties of concrete control will be practiced without fail.

In connection with joint spacing, volume change, and the multitude of details that attend the raising of dams such as Assuan Dam, in Egypt, and O'Shaughnessy Dam, in California, there has been some very interesting and comprehensive investigation into the realm of actual construction joint performance.

After making a comprehensive survey of cracks in twenty-one dams in the western part of the United States, H. M. Westergaard,[40] M. Am. Soc. C. E., states that "By observing cracks and joints in a number of dams one is impressed with a certain regularity of the phenomena" but adds that "No single distance can be stated defining the spacing of the vertical expansion joints necessary to avoid major cracks between them. As a general rule, a smaller height of the dam requires a closer spacing."

The trend toward closer spacing of joints is not confined to concrete dams; it is also evident in concrete pavement construction. E. F. Kelley, chief, Division of Tests, U. S. Bureau of Public Roads, concludes[41] that "Reasonable assurance of the absence of the transverse cracking in concrete pavements can be obtained only by the use of slabs that are about square with dimensions of the order of 10 feet."

In regard to the spacing of longitudinal joints in a dam, the same principles apply that have been discussed for transverse joints. From the standpoint of structural sufficiency, longitudinal cracks in a dam are far more objectionable than transverse ones, since the latter principally affect the watertightness qualities of the dam, whereas the former may divide the dam into parts that would not be stable, especially if hydrostatic pressure builds up to reservoir head in the crack. Longitudinal joints have been used in the construction of a few dams such as Assuan, Boulder, and Shasta, but the intricacies forced into the construction procedure because of the necessity for grouting longitudinal joints have made this type of joint relatively unpopular and one that should be avoided, if practicable.

In a straight gravity dam the opening of longitudinal joints, or the formation of irregular longitudinal cracks where there are no longitudinal joints, may or may not be a serious problem, but it is certain that such openings should be

[40] "Cracks Observed in Dams," by H. M. Westergaard, *Technical Memorandum* of Bureau of Reclamation, July 8, 1930.

[41] "Application of the Results of Research to the Structural Design of Concrete Pavements," by E. F. Kelley, *Journal*, Am. Concrete Inst., January, 1939, p. 437.

avoided, if possible, in the interest of the permanent safety and integrity of the structure. Where at all practicable, it would appear preferable, even at considerable additional expense, to construct a straight gravity dam without longitudinal joints. The production of an ideal uncracked monolith is possible if the temperature of the material of which it is constructed is controlled so as to avoid initial high temperatures and high temperature differentials in short distances. The latter condition starts cracks at the surface, which, once started, are as difficult to stop as the crack in a windowpane.

The construction procedure followed for crack elimination at Hiwassee Dam, in North Carolina, and the results produced are encouraging. Hiwassee is the first dam in which a combination of crack-elimination methods of construction has been followed consistently. Although some cracking at horizontal-lift joints was noted during the winter of 1940, the writer is advised that no vertical cracking on the faces of the dam has yet been experienced and that longitudinal cracking occurred in only a few monoliths.

Shallow lifts, proper type of cement, low-cement content, refrigeration of mixing water, sprinkling the coarse aggregate and blowing air through it to standardize moisture content and reduce temperature, restrictions on placing concrete during hot weather, sufficient time between lifts to permit a large amount of heat loss from the surface of the lift, and cooling with embedded pipes are the principal means of avoiding or minimizing the tendency to crack. All of these factors affect the cost of the finished concrete and hence must be weighed carefully as to cost and benefit before deciding upon specification limits for each factor.

Foundation conditions also may play a very important rôle in the formation of cracks. Each and every monolith in a straight gravity dam should be designed to stand as an independent unit on its own foundation. As the height of such dams increases, there appears a "no man's land" where there is an ever-present danger of longitudinal cracks parallel to the axis of the dam. This situation can be handled in one of three ways: (1) The monolith can be built so as to prevent the formation of temperatures within the mass that will form cracks during the cooling period; (2) the monolith can be built in columns with joints parallel to the axis of the dam that can be grouted after temperature stabilization; or (3) the monolith can be built with closing slots that can be filled with concrete after the surrounding mass has cooled naturally or has been cooled artificially. Obviously, the grouted longitudinal joint has some very definite structural weaknesses which expose it to possible future cracking and its attendant hazards; so has the slotted type of joint, but to a much lesser degree, and this weakness can be largely eliminated if reinforcement is extended into the slot from each side of the joint so as to make an interlocking reinforced mass of the closing plug and adjacent concrete. The proper form for this slot, the treatment of the surfaces of the concrete in the slot before the plug is poured, the details of the reinforcement, the season of the year and rate at which the plug is poured, the characteristics of the plug concrete as compared to the adjacent concrete, and many other minor points vitally affect the transfer of stress across an area which at best is not possessed of the same mass characteristics as the monoliths which it unites. Hence, it is firmly believed that the artificial

control of temperatures as contemplated under item (1) is the most satisfactory, probably the only satisfactory, solution to a problem that confronts the builders of all high, straight gravity dams.

Foundations with faults or planes of weakness that might produce cracks parallel to the axis of the dam after the temperature has stabilized within the monolith are possible but not common and would require special design consideration which cannot be covered in a paper of this length.

Plain Versus Keyed Joints

Joints in dams are either plain or keyed and, if keyed, there may be few or many keys. The keys may be narrow or wide and shallow or deep, or a combination. Joints in which the keys are so dimensioned as to give a "fifty-fifty" theoretical shearing resistance are quite popular with many design agencies. The batter on the sides of the keys also is a point on which there is a wide range of preference, the batter varying from nearly perpendicular to the plane of the joint to as flat an angle as 30° off the plane of the joint. One has only to examine a few sets of drawings to be convinced that there is no particular "rhyme or reason" to keys and keyways, and that the designer's fancy has been given full expression in this respect.

As a deterrent to the passage of water, keys and keyways in the vertical transverse construction joints in dams may have some desirable features, since a keyed joint with slight batter on the sides of the keys will silt up more readily than a plain joint because the actual width of opening at the battered section is only a fraction of the width of the plain joint.

Obviously there is less likelihood of cracks starting with the plain joint than with the keyed joint since the reentrant angles in the latter are so often the starting points for cracks that develop due to surface differential temperatures during the construction period but later are the entering wedges for objectionable cracking parallel to the axis of the dam that might otherwise never develop.

It is believed that the keyed joint in concrete dams and retaining walls is a hang-over from the mortise and tenon so commonly used in timber construction. Theoretically, it may be relatively easy to justify the use of keyed rather than plain joints, but if a careful analysis is made of each case in question it is believed that the questionable features of many keyed joints would outweigh the unquestionable ones and that many keys now in common use would be omitted.

The question as to the desirability of keys in horizontal joints in concrete is an old and much discussed problem that has not yet been settled to the satisfaction of all, although it is now generally conceded that the elimination of keys on the horizontal lifts of mass concrete is desirable. With vibration, modern concrete control, and strict inspection, the leaky horizontal joints of the past have largely disappeared. It is now possible to secure good bond on nearly all horizontal surfaces, and hence the apparent need for some kind of an obstruction to the flow of water along the plane of the joint has disappeared. As for shear, it is believed that a rough, well-bonded surface is far superior for the transfer of stress than dependence on keys.

The proper kind of concrete immediately below the plane of the joint is still lacking in too many horizontal joints in thin walls. This is due primarily to working the surface layer of concrete too much, the result being a layer of concrete of high water-cement ratio and low density and strength. To attempt to work a horizontal key form block down into such a surface aggravates the matter still further and serves only to make bad matters worse. Hence it is believed that, if horizontal joints in thin walls were treated in the same manner and with the same care as horizontal joints in mass concrete dams, corresponding results would be obtained. If some form of stop or key is insisted upon for

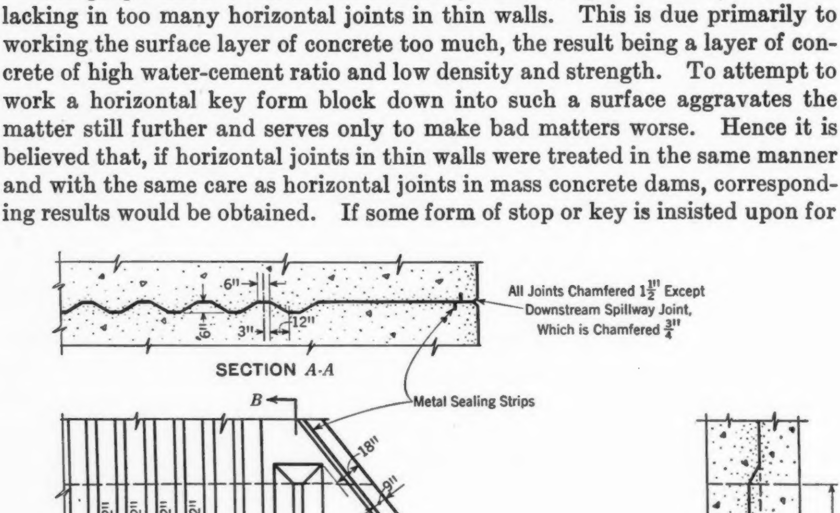


FIG. 10.—TYPICAL CONSTRUCTION-JOINT DETAILS FOR DAMS OF THE BUREAU OF RECLAMATION

horizontal joints in thin walls, a ¼-in. steel plate 6 in. wide set across the joint will accomplish the desired result with the least disturbance to the finished concrete surface. It will then be much easier to secure satisfactory concrete below the joint, and to clean up the joint surface just before the next pour is started, than with a key. The less a concrete surface is disturbed after the concrete is placed and consolidated the better. In the final cleanup before concrete placement is resumed, wet sandblasting is the most effective method of producing a satisfactory surface on which to deposit concrete and secure good bond. Careful inspection will always be necessary on this class of joints to prevent rock pockets at the surface of contact and in the corners. A half-inch layer of mortar of the same properties as exist in the concrete will be helpful in securing good horizontal joints. This mortar, however, should be deposited immediately ahead of the batch of concrete so that it will not stiffen up and prevent the coarse aggregate particles from bedding satisfactorily.

Dams such as Boulder, Grand Coulee, and Shasta bring into the construction joint feature many problems aside from strictly technical ones. In the construction of Boulder Dam, the details shown in Fig. 10 were developed for

the radial joints. They proved very satisfactory and have likewise been used in the construction of Grand Coulee, Shasta, and other dams. Although the forming of keys is only a detail of form work, it is one of the details that affects the cost quite materially if it is not standardized to the point where rapid shifting of forms is permitted, and re-use with long life is guaranteed. The details shown in Fig. 10 may appear rather complicated, but this key system has proved to be more satisfactory than any other system with which the writer is familiar. Similar form details[42] were also used in the construction of Seminoe Dam, in Wyoming.

Slots Versus Grouted Joints Versus Open Joints

An ideal dam should have neither transverse nor longitudinal joints or cracks. Whether this ideal is possible of attainment is open to question due to the fact that volume change in the concrete of the dam and volume change in the foundation supporting the dam are possibilities that are always present.

Whether the volume change contemplated in the preceding paragraph and the large variety of cracks it produces are so objectionable as to warrant expensive construction procedure to avoid them is also open to question. However, there is no question but that the ideal crackless dam (especially as regards longitudinal cracks) is preferable and is worth considerable study to attain in so far as practicable.

In recent years, since volume change in concrete has been better understood, there has been a concerted effort by many agencies to render both arch and gravity dams more nearly monolithic than has been their condition in the past, in an attempt to obtain better distribution of stress and closer agreement between analytical assumptions and actual conditions. This has been accomplished through the use of closing plugs between monoliths, or the grouting of the joints between the monoliths after the latter have cooled to approximately minimum annual temperature. Closing plugs, or slots, are ordinarily 3 ft thick and filled with concrete similar to the concrete in the adjacent monoliths.

The necessity for grouting the transverse joints in a straight gravity dam appears questionable. Instead, it would appear that each monolith should be stable on its own base. To attempt to modify the distribution of stress in a straight gravity dam by grouting the transverse joints appears to have too many questionable features to warrant the practice.

In a curved gravity dam, and in arch dams, conditions may be somewhat different from those in a straight dam, and it may be economically desirable to take advantage of the transfer of stress from gravity to arch elements. In that event, the grouting of joints or the use of closing slots is vital to the theoretical accomplishment of the stress transfer if, for instance, tension cracks near the base of the dam on the upstream side are to be avoided. Either method will yield the desired results if satisfactory construction and grouting details are developed. There are advantages in both systems and hence both systems should be given due consideration. The slotted method of treatment seems to be favored in foreign countries, whereas joint grouting is the system preferred in the United States. Flood diversion requirements during construction may

[42] *Western Construction News*, March, 1939, p. 98.

render the use of slots with arch dams impracticable because of the possibility of sudden filling of the reservoir and the collapse of the unsupported arch monoliths.

In commenting on this subject, A. Thimel (director general of the Department of Public Works, in France) concludes[43] that for arch dams "a system of slots with keys is best to ensure the bonding of the blocks," and that for curved gravity dams "The system of radial slots with keys is preferred * * * and shrinkage joints become unnecessary." He also draws the conclusions from experience to date "that only a few elementary and now well-defined precautions are required when designing joints of a large dam" and that "if these precautions are taken, the joints will never be one of the weak points of the dam." The latter statement is accepted heartily; and in so doing the writer has in mind one dam in the United States which is an excellent example of satisfactory and watertight concrete, but the joints were spaced so far apart that the objectionable cracking and leakage have overshadowed the other good qualities entirely.

In describing the Mareges arch dam, A. Coyne (chief engineer of bridges and highways, in France) states[44] that "the dam was divided into blocks separated by slots one meter wide, and that to insure watertightness copper stops were installed at the upstream face and a system of horizontal and vertical grout pipes installed, and the joints between the slot and the adjacent blocks grouted." In the United States at Boulder Dam under similar circumstances the 8-ft cooling slot in the middle of Boulder Dam was filled with concrete, after the dam had been cooled to its mean annual future temperature, without grouting the joints on either side and with no apparent leakage or other undesirable effects.

At the time the contract was let by the Bureau of Reclamation for the construction of the Gibson Dam, in Montana, the plans provided for closing slots or plugs the full height of the dam between certain pairs of monoliths, these to be later filled with concrete. Subsequently, this plan was abandoned and the joints grouted by means of the pipe system which has since been adopted as standard practice by that Bureau. Since the joints in Gibson Dam were grouted, Deadwood, Owyhee, Cat Creek, Boulder, Parker, Madden, Grand Coulee, and Seminoe dams have been treated in much the same way. With the joint surface clean and suitable for the adherence of a grout film, there does not appear to be any valid reason why the radial grouted joint in the interior portions of the dam should not be permanently satisfactory, provided the grout film is introduced under sufficient pressure to drive the excess water into the pores of the adjacent mass concrete so as to create a dense, durable, and impermeable film. Grout films not produced under pressure are likely to be chalky and to disintegrate easily.

Thin arches, which will cool to mean annual temperature throughout the entire mass during the construction period, would seem to offer the most at-

[43] "Joints de Retrait et Joints de Contraction et Dilatation," by A. Thimel, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 61.

[44] "Clavage des Barrages Voûtes," by A. Coyne, *loc. cit.*, p. 91.

tractive field for the slot rather than the grouting method of joint treatment, since the constant change in surface and near-surface temperatures may, in time, affect the grout film adversely by rupturing its bond with the adjacent concrete, thus contributing to its ultimate disintegration and the consequent increase in arch stress due to decreased arch thickness.

Decreased arch thickness, as noted in the preceding paragraph, and the decrease in arch thickness resulting from the installation of grout stops, are not serious matters for arches of appreciable thickness since the worst that will probably happen is that internal arch stresses will be increased and the distribution changed somewhat, all of which can be taken care of by the normal or usual factor of safety. In thin arches, or in the upper part of arch dams in which this decrease is an appreciable part of the total arch thickness, provision should be made for increasing the total thickness so as to provide for satisfactory stresses in the part of the arch joint that actually remains in contact under maximum load conditions.

When flood diversion during construction is adequate, and where time for the cooling of the blocks or monoliths of the arch permits, it is believed that due consideration should be given to the use of slots or closing plugs instead of grouted joints. Under this method of construction, there will result a mass of concrete that is more nearly permanently monolithic than under the joint grouting system where constant change in volume near the surfaces of the dam may ultimately break down the bond between the grout film and the adjacent concrete and thus gradually result in the disintegration of the film. This may or may not be of any very great moment, but, if the film is not as permanent as the adjoining concrete, it is not worth putting in the joint in the beginning. The interior of the dam, beyond the effect of annual temperature cycles, however, is not subject to the foregoing criticism and, if satisfactorily grouted, the film should be as permanent as the adjoining concrete.

However, for the surface layer of concrete in any type of dam in which there is a daily and annual temperature fluctuation, the permanency of a thin film of grout in the joint is open to serious question, and the possible ultimate disintegration of this film would lead one to conclude that the slot method of joint construction might be more permanent and satisfactory than the grout-film method.

Joint Surface Treatment

Where future differential movement of the surfaces of a vertical joint parallel to the plane of the joint is anticipated, the use of some form of asphaltic or bituminous compound with or without asbestos fiber may be desirable; but even here the indiscriminate use of asphaltic or bituminous paints and putties on joint surfaces is not founded on necessity. If the keys in a joint are properly shaped or tapered so as not to "hang up" when the adjacent surfaces of a block separate, due to contraction, there is little need for any kind of surface treatment. However, if movement is anticipated between joint surfaces parallel to each other, the keys and keyways near the outside surface of the structure may give trouble by causing concentration of stress that will produce cracks. This type of possible movement calls for adequate study and in certain cases for

provision for surface treatment at critical points and possibly for a compressible filler where keys might shear out slabs of the surface concrete.

Bonding Joint Surfaces

The adequate bonding of horizontal and inclined concrete joint surfaces has been one of the most difficult problems to solve satisfactorily that has been encountered in the concrete manufacturing process. Bonding vertical surfaces is not as common a requirement as for inclined and horizontal surfaces, but the same principles apply and, where necessary, are more easily applied.

During recent years horizontal joint cleanup with a high-velocity, air-water jet, applied during the period between initial and final set, has largely displaced other methods of joint surface treatment, and when applied at the proper time and with the correct technique has produced excellent results.

Before the introduction of vibration, the roughing-up of joint surfaces (covering them with a coat of grout and then depositing thereon concrete of a little higher slump than later used for the overlying mass) was the accepted practice. Since the development of the vibrator and the acceptance by the construction industry of the practicability of placing 1-in. to $1\frac{1}{2}$-in. slump concrete, many of the objectionable features of horizontal concrete joints have disappeared. Cores drilled across these joints reveal a more satisfactory density and a greater uniformity of aggregate distribution than formerly, without so much of the old-time tendency toward concentration of a matrix with too high a water content immediately below the lift joint.

Metal strips at horizontal construction joints, raised or depressed keys, and elaborate cleanup requirements are gradually disappearing from construction practice. However, in Claytor Dam in Virginia, completed in 1939, the horizontal construction joints were provided with a raised key (24 in. by 6 in.), a copper-bearing steel plate water stop ($\frac{1}{8}$ in. by 10 in.), and a formed open drain (8 in. deep by 10 in. wide), the center lines of which three items are distant 3 ft, $5\frac{1}{2}$ ft, and $7\frac{1}{2}$ ft, respectively, from the upstream face of the dam. The longitudinal drains stop short of the transverse contraction joints between blocks and are connected to the interior drainage system of the dam. This design of the joint itself is accompanied by a joint-treatment specification requiring the entire surface of each horizontal lift to be "cut back" a minimum depth of 1 in., and, for a strip 18 in. wide around all permanently exposed faces and the upstream face of the dam, the "cutback" must vary between 2 in. and 5 in. in depth, depending on the slump and the height of lift. The "cutback" normally was accomplished prior to initial set with air or water jet, or both, under pressure, or by shoveling. No "cutting back" was permitted between the time of initial and final set, "cutting back" after final set being accomplished to the prescribed depth by the use of hand-picks or equivalent means. Thus it is apparent that the telltale examples of past poor construction practice have not yet been forgotten by some agencies and that this phase of construction procedure is still of major concern if satisfactory long-time performance is earnestly desired.

Observations over a long period of years, supported by recent tests by the Bureau of Reclamation, indicate that the less freshly placed concrete is disturbed after consolidation and prior to hardening, the better the qualities of

bond and watertightness will be, and that the only cleanup necessary for a properly designed mix, properly placed, is that done just prior to the placing of another layer of concrete. After a thorough study of this phase of concrete placement, R. F. Blanks (engineer, Bureau of Reclamation) concludes[45] that the most desirable results with horizontal joints will be obtained if the following general rules are adopted:

(1) Do not permit placing concrete which bleeds and forms laitance excessively.

(2) Avoid excessive working of fresh concrete at the construction planes.

(3) After the concrete is once placed, do not permit it to be disturbed until it has attained sufficient strength to withstand traffic—omit the initial cleanup and keyways.

(4) Keep the surface continuously moist until the next layer is placed or until sufficient hydration of the cement has been obtained (where practicable, wet sand as a curing medium is preferable to any other).

(5) Clean the old surface thoroughly just prior to placing the next lift. The most satisfactory method is by wet sandblasting and washing. If the surface is essentially free of laitance and coatings, good results can be obtained by thorough wire brushing and washing. Where practically ideal placing has been obtained and the time interval before placing the next lift is not more than three days, high-velocity water jets may be sufficient. Chipping should be used only in extreme cases where damaged or defective areas are encountered.

(6) Thoroughly and effectively, broom into the old surface a layer of suitable mortar and place new concrete immediately.

The *Concrete Manual* of the Bureau of Reclamation illustrates[46] the condition of placement at a horizontal joint that is responsible for a great deal of the lack of watertightness in concrete and the characteristic surface disintegration found at and immediately below the horizontal lift joints in so many hydraulic structures.

In 1939, Charles E. Wuerpel[47] (engineer, Central Concrete Laboratory of the North Atlantic Division, U. S. Engineer Department, U. S. Military Academy, West Point, N. Y.) discussed and illustrated the results of a series of freezing and thawing tests of sixty-seven cores drilled across horizontal construction joints. It is interesting to note that, in some of these cores, although the joint itself remained sound, the deterioration immediately below the joint was much greater than in the remaining portion of the core. This indicates that, although the horizontal construction-joint cleanup and bond are being taken care of satisfactorily in the more recent hydraulic structures, there is still too much of a tendency to let an inferior grade of concrete accumulate in the upper portion of a concrete lift. For this reason it is believed that Mr. Blanks' rule (2) cannot be overemphasized and that some additional research along the

[45] "Treatment of Horizontal Construction Joints in Concrete to Improve Bond, Shearing Resistance and Watertightness," by R. F. Blanks, *Memoranda* to the Chf. Designing Engr., Bureau of Reclamation, March 8 and May 28, 1938 (not published).

[46] *Concrete Manual*, Bureau of Reclamation, U. S. Dept. of the Interior, March, 1939, Fig. 68, opposite p. 210.

[47] "Tests of the Potential Durability of Horizontal Construction Joints," by Charles E. Wuerpel, *Journal*, Am. Concrete Inst., January, 1939, p. 181.

line of Mr. Wuerpel's experiments would reveal some very definite trends that are worth noting for future durability.

WATER STOPS

An absolutely watertight permanent membrane of appreciable thickness would be desirable on the upstream face of any dam, but whether it is necessary and worth the cost is open to question.

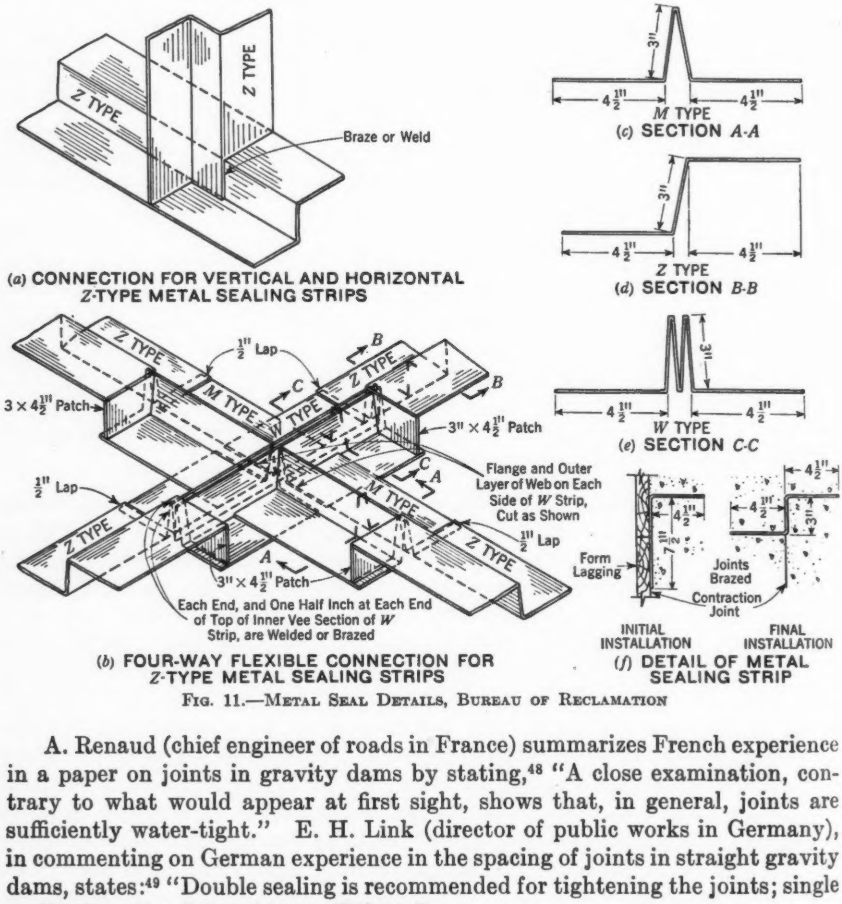


FIG. 11.—METAL SEAL DETAILS, BUREAU OF RECLAMATION

A. Renaud (chief engineer of roads in France) summarizes French experience in a paper on joints in gravity dams by stating,[48] "A close examination, contrary to what would appear at first sight, shows that, in general, joints are sufficiently water-tight." E. H. Link (director of public works in Germany), in commenting on German experience in the spacing of joints in straight gravity dams, states:[49] "Double sealing is recommended for tightening the joints; single sealing has not always been sufficient."

In the original outline of this paper, it was planned to prepare schematic and enlarged detail plans to illustrate typical water-stop installations in dams in various countries, so that their relative dimensions and peculiarities could be

[48] "Les Joints des Barrages-Gravité," by A. Renaud, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 19.

[49] "Entstehung und Abdichtung von Schwind-, Zusammenziehungs- und Dehnungsfugen in Staumauern," by E. H. Link, *loc. cit.*, p. 109.

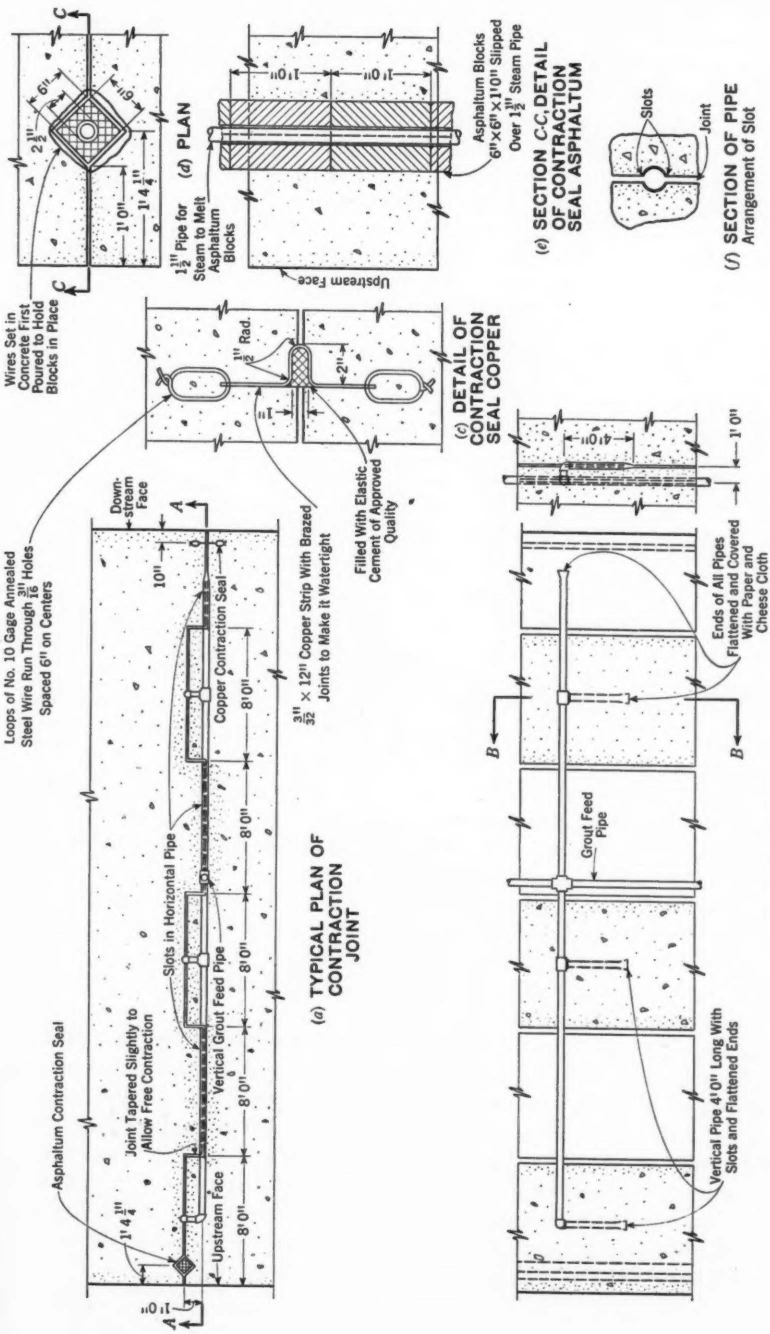


FIG. 12.—CONTRACTION-JOINT DETAILS FOR CUSHMAN DAM No. 1

readily visualized. This would have made an interesting comparative picture, but, after getting the joint detail descriptions translated and making several layouts, it was apparent that individual fads and fancies had so permeated this realm that it would be very difficult to produce typical illustrations which would do justice to dams in general. Hence, it was decided to summarize instead of illustrate, and with this in mind, a few of the outstanding water-stop features of typical dam construction jobs are reproduced in Figs. 11, 12, and 13.

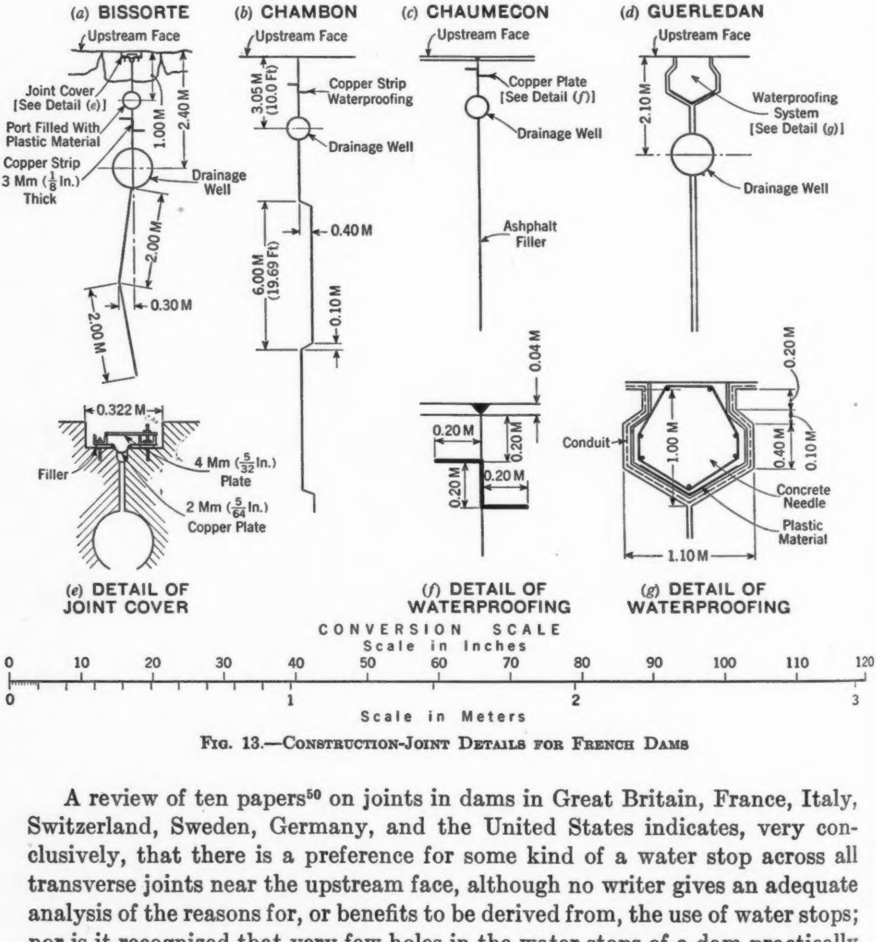


FIG. 13.—CONSTRUCTION-JOINT DETAILS FOR FRENCH DAMS

A review of ten papers[50] on joints in dams in Great Britain, France, Italy, Switzerland, Sweden, Germany, and the United States indicates, very conclusively, that there is a preference for some kind of a water stop across all transverse joints near the upstream face, although no writer gives an adequate analysis of the reasons for, or benefits to be derived from, the use of water stops; nor is it recognized that very few holes in the water stops of a dam practically eliminate the structural benefit. Data on water-seal details in various dams are assembled in the Appendix. Copper for water stops seems to be preferred to any other metal by the designers of all countries.

[50] Question IV, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 3 *et seq.*

In commenting on contraction and expansion joints in a power flume, Hugo Flodin (civil engineer at the Royal Board of Waterfalls, in Sweden) states[51] that "to prevent leakage and avoid electrolytic action, lead sheets have been placed at each side of the copper sheeting." This is the only reference, in the aforementioned ten papers on construction joints, to possible electrolytic action in connection with the use of copper for water stops, although it has been known for many years that such action is within the realm of possibility. Just how permanent the copper stop is as a water seal will probably never be known, because few dams will ever be torn down and the water seal examined in detail to determine its qualities of watertightness and durability. The writer is inclined to the opinion that, as a watertight membrane, a copper stop is open to serious question and at best is merely a deterrent to the passage of water through the joint. This should not be taken as a condemnation of metal water stops, but rather as a word of caution against placing too much confidence in the watertight membrane conception of a metal stop and in its permanence, especially where structural sufficiency might later be affected adversely by leakage due to its ultimate deterioration. After all, even if the copper stop remains tight at its many seams, the concrete into which it is set is not watertight, and the path of percolation around the stop is all too short. Of the available methods of limiting objectionable leakage through the joints of a dam, it is believed that metal water stops near the upstream face are least permanent and that where any and all leakage must be prevented consideration should be given to grouting the joints or to closing plug joints (slots). Possibly a joint drainage system, so common in European dams, should be installed.

In the United States, it has become nearly standard practice to use 20-oz soft annealed copper for water stops. In Boulder Dam, however, the water stop selected was monel metal because of its record for durability. In other countries also, copper plate seems to be preferred for nearly all installations where a water stop appears desirable, although there are occasional installations where monel metal or stainless steel is used. The thickness shown on most foreign illustrations available is 2 mm, which is nearly three times the thickness of 20-oz copper (0.027 in.). The shape of the stop and the length provided for embedment, however, are so variable that it is difficult to summarize satisfactorily. Practice in other countries seems to be to place the water stop much farther from the upstream face than is customary in the United States. Most of the installations in the United States are only about 12 in. from the upstream face. It is believed that much more satisfactory and permanent results would be obtained if water stops were set far enough back from the surface to be beyond the zone of appreciable temperature variation, which is undoubtedly detrimental to the bond between the concrete and copper.

In the United States, the **Z**-stop seems to have proved more satisfactory and serviceable in actual use than any other type. This type, however, does not fit all conditions and hence the **M**, the **U**, the **V**, and many other modifications, as illustrated in Figs. 11 and 12, have been developed for special jobs where the joint movements are rather complicated. Whether annealed copper will retain the qualities imparted by the annealing process, permanently, where

[51] "Contraction and Expansion Joint in Intake Flume," by Hugo Flodin, *Transactions*, 2d Cong. on Large Dams, International Commission on Large Dams of the World Power Conference, Supt. of Documents, Washington, D. C., 1938, Vol. III, p. 188.

the joint is subject to constant change in width due to temperature variations, or will ultimately become brittle and crack, is also open to question, especially in joints where the copper stop is merely crimped and the opening and closing of the joint produce an appreciable movement in the crimped bend. Water and grout stops should be jointed by brazing rather than soldering since the latter method has not proved nearly as satisfactory as the former.

Some kind of bead or mechanical bond at the embedded end of the stop seems imperative to assist in preventing bond failure. This is rather difficult to secure if the copper is purchased in rolls and cut to lift length on the job. The same result as with the bead is accomplished, however, if the edge is snipped every few inches and alternate sections bent to form a short **L**, or if bond holes are punched in the copper near the outside edge. A mechanical bond of some kind is believed more important than increased length of embedment, especially if the difference in coefficients of expansion of copper and concrete is given any weight as affecting the validity of the bond where temperature ranges in the surface concrete surrounding the copper stop may be as much as 50° F annually. One of the construction problems that is difficult to solve satisfactorily is to attain 100% bond in the embedded part because it means eternal vigilance throughout the construction period. Rock pockets and big slugs of concrete always seem to happen at the wrong time and in the wrong place to produce a watertight water stop. Wherever watertight joints in massive structures are necessary, it is believed that joint grouting after minimum volume is reached will give more satisfactory results than dependence on metal stops whose function can be so easily circumvented if rock pockets or poor bond develops around the embedded legs of the water stop—as so often happens, especially on the blind side of the joint.

In Fig. 13 are shown the essential details of various transverse joints that have been used in France and other countries in Europe. This construction appears to have considerable merit from the standpoint of durability and watertightness, and it is believed that it could be used to advantage in reduced section in many places.

Undoubtedly, the metal water stop is a necessary adjunct of the construction joint in thin concrete structures; but its desirability is questioned for massive structures which, once cooled to mean annual temperature throughout, are subjected to no further volume change at depths not affected by annual temperature variations. It would seem that either the grouting of the construction joints or joint construction embodying slots, which seem to be quite popular in foreign countries, is a better, more permanent, and more satisfactory means of rendering a construction joint area tight than dependence on a thin metal strip. Any imperfection in the metal stop, or the concrete adjacent thereto, from any cause whatsoever, exposes the entire joint area to any objectionable effects that water may have on the joint. The outstanding example of a slot or a closing plug with few, if any, objectionable results is the cooling slot in the center of Boulder Dam. It is 8 ft wide and was filled with concrete at a fairly rapid rate after the cooling of the dam was completed. Subsequently, cores drilled across the contact surfaces of the plug and the dam did not develop any reason to question such methods of construction procedure.

Grout Stops

The grout stop is an outgrowth of the water stop, its function being the same, but the required period of serviceableness being entirely different. Hence, on some dams, there have been developed, for joint grouting purposes, entirely different types of stops. On Boulder Dam, for instance, the grout stop consisted of a straight copper strip coated with asphalt mastic and embedded across the joint, the mastic serving to maintain grout tightness and still permit opening of the joint without rupture of the metal stop. The kind of metal that should be used in a grout stop is of relatively little importance since permanency is not one of the requirements.

The proper location of a grout stop is a moot question. If the grout film near the surface of the dam was a permanent and durable filler, the closer the grout stop could be located to the surface of the dam the better. However, since the durability of the grout film near the surface appears so questionable, it would appear that in the interests of permanence for the film the grout stop should be set well back from the surface. Temperature variations at 3 ft from the surface would certainly be much more conducive to permanence in a grout film than at 9 in. It is noted that the tendency in foreign countries is to place grout and water stops farther from the face of the dam than in the United States. Grout stops placed too close to the surface give trouble during the grouting operation because the concrete cantilever beam supporting the stop fails, due to grout pressure from the interior of the joint, and a sliver of concrete adjacent to the joint is pushed off.

Joint Fillers

The indiscriminate use of joint fillers—a design and construction practice that is on the increase in the United States—is objectionable from the standpoint of appearance; and in many cases there is no basic reason for their use. The necessity for a joint to eliminate an unsightly crack does not carry with it the parallel necessity for some kind of joint filler. These are two entirely different problems—one involving tension and the other compression. Even where a joint filler is indicated as necessary, a relatively thin, inconspicuous filler may answer the purpose just as well, or even better, than the thick one which is such common practice.

Thin slabs and some thin walls require suitable provision for expansion without buckling or spalling if the temperature differential to which they are subjected is rather high and the structure is not restrained. However, there is still too much of a tendency to use thick joint fillers and wide joint spacing, rather than close spacing of joints; and to use the minimum thickness of filler and a slight chamfer, instead of a filler, wherever it will accomplish the desired purpose—that is, prevent spalling at the surface.

During recent years the writer has made a special point of observing and photographing construction joints in concrete structures with a view to determining just where a joint filler is needed and where a chamfer only is indicated. As a result of these observations, it is concluded that many joints that should have spalled, because of the absence of adequate joint treatment, did not, and most of those that did show some spalling did not need a joint

filler but should have had a small chamfer only. It was also observed that the fillerless joints were much more pleasing to look at than the wide joints that had big blotches of joint filler protruding.

Fig. 14 shows a vertical construction joint in the parapet on a curved gravity dam which is typical of the entire dam in regard to the result of ten

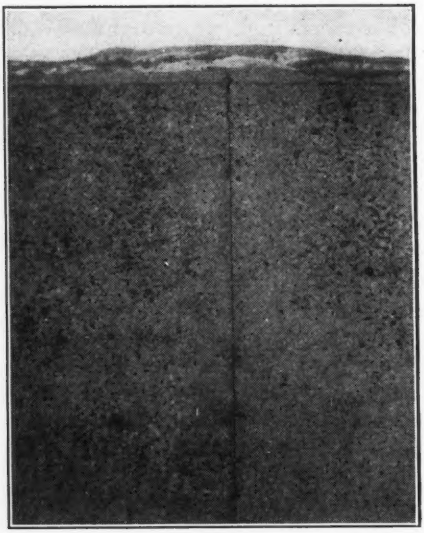


Fig. 14

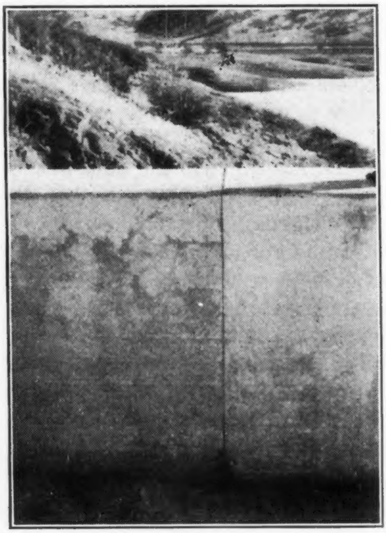


Fig. 15

years' exposure to the elements in a moderate climate, ranging from a few degrees below freezing to 110° F. This joint does not contain a filler and was not painted. A half-inch chamfer at this point, as in most other similar joints on both parapets, would have sufficed to prevent what little spalling has occurred. The sun strikes the downstream face of this dam during the greater part of the day, and as a result the crest of the dam at the center deflects upstream every summer $\frac{3}{4}$ in., regardless of the level of the water in the reservoir; yet there is little or no evidence of surface compression spalling on joints that were not chamfered.

Fig. 15 is also typical of the joints in the parapet on a straight dam in which the joints were neither treated with a filler nor mastic of any kind; nor were they even painted. Here, too, the result would have been more pleasing to the eyes if a slight chamfer had been used, but there is certainly no indication of the necessity for such joint fillers as is seen in so many modern designs.

Thick straight walls, dams, and thick slabs anchored to rock do not require joint filler if properly chamfered, since in so doing the surface layer of concrete is relieved of stress from contact across the joint and the thrust due to expansion is distributed over sufficient cross-sectional area within the mass to reduce the stress at the surface to limits well below the point of rupture.

In dams and in thick walls and slabs the initial temperature due to heat of setting of the cement will rarely ever be attained thereafter throughout the

mass, and hence, if the possible surface spalling at the joint is provided for by proper chamfering, no further consideration need be given to the remainder of the joint surface. Even the painting of the joint to prevent possible bond is unnecessary and is not practiced in dams where joints are to be grouted in the interests of rendering the structure more nearly monolithic. Hence, if not needed for joints that are to be grouted, it certainly is not needed for the average mass concrete joint surface.

Joint Grouting

As has been noted previously herein, the successful grouting of the joints in a dam is a highly specialized line of work requiring adequate equipment and a carefully developed and proved technique. The ideal joint-grouting job would be one that would permit the simultaneous grouting of all joints in the dam for their full height. This procedure, however, is rarely possible unless the dam is limited as to height and length. The very nature of the work requires careful advance planning and the checking of every part of the grouting system prior to starting the grouting operations, because, once started, it is difficult, if not impossible, to stop long enough to correct defects or secure additional equipment.

The greatest detriment to successful joint grouting is leakage around or through the grout stops, especially the horizontal runs. A nail hole in the copper, a small break in the soldering or brazing at a joint, or a rock pocket on the blind side of the joint may make it impossible to build up any pressure in the joint or even to fill the joint completely with grout of the proper consistency. This is due primarily to carelessness during the process of installing the grout stop; but the grout stop is just another one of those details that cannot occupy "the center of the stage" if the economics of concrete yardage placement is also striving for the "limelight," as it usually is.

Leakage affects the quality of the resulting grout film, the proper sequence of joint grouting operations, the drainage system, the uplift measurement experimental equipment, if any, and last, but not least, the structural sufficiency of the grout film.

In the grouting of the vertical radial joints in a high arch dam in a narrow canyon in which the grouting operations must be handled in lifts and the grouting of every lift tends to open up a V-shaped opening in the lift below, the horizontal sections of the dam occupied by these V's are prevented from taking their proper proportion of arch stress and consequently their share of the load is shifted elsewhere. Thus it would be possible with considerable abutment yielding during the grouting operation to decrease the bearing area between voussoirs and thus increase the arch stresses to undesirable limits. Hence, it must be realized that grouting joints in dams, to accomplish or even to approach a better distribution of stress than would be obtained in an ungrouted joint, is a highly specialized job that requires experience, a carefully planned method of procedure, an adequate organization, and equipment suitable for that particular class of work.

Cores drilled across grouted joints have demonstrated rather conclusively that grout films of satisfactory density can be secured and also that joints

which have reopened may be regrouted successfully; but whether the conception of "monolithicness" popularly held is generally achieved or is even a "fifty-fifty proposition" is open to question. The fragility of the film and its susceptibility to bond rupture, if the joint space is not completely and permanently filled, cannot help but influence any one who analyzes a dam in all its minutiæ of design and construction detail to carefully weigh for future construction the relative value of grout versus slot type of joint construction.

Before starting on the extensive program of joint grouting which the Bureau of Reclamation has done in recent years, a comprehensive series of laboratory tests was conducted with a 5-ft by 7-ft split concrete cylinder in which the width of the joint opening, the consistency of the grout, and the application of pressure were varied through wide limits. As a result of this series of tests, a joint-grouting technique was perfected, which, with the changes and improvements normal to field progress on all such work, has given very satisfactory results as far as all outside appearances are concerned. Whether the joint coverage approaches 100% and whether each joint is completely filled at the time the grouting operations are completed are factors probably open to speculation. That a dense grout film of low water content can be produced with moderate pressures is a fact supported by diamond-drill records and experimental data.

Experience to date seems to indicate rather conclusively that, in order to grout a joint satisfactorily and be able to take care of emergencies when plugged pipes and joints occur, it should be possible to circulate the grout in the joint. The system should also be designed so that it will be possible to weep off thin and foamy slugs of grout mixture and, when the joint has been filled with grout of uniform consistency, to apply a reasonable amount of pressure (25 to 50 lb per sq in.) so as to drive excess water in the grout film into the adjoining concrete and decrease the water-cement ratio of the film, thus increasing its strength and other desirable qualities. With the cement screened through a 200-mesh sieve, the joint-grouting operation generally can be accomplished by starting with a water-cement ratio of 1.0 and reducing as soon as conditions warrant to a ratio of 0.75, more or less. This procedure, of course, assumes proper preparation of the joint prior to grout injection.

The pressure that is applied to drive the surplus water out of the grout film and into the adjacent concrete has a very pronounced effect on the quality of the film and hence should be given as much consideration as is practicable under the pressure limitation dictated by the structure and the site. The flowage properties of the cement grout are quite materially affected by the coarseness of the cement. Cement screened through a 200-mesh screen was used in the grouting of Owyhee and Boulder dams and has a very definite advantage over unscreened cement, since unscreened cement grout causes more or less plugging in the pipes and fittings, will not stay plastic as long as screened cement grout, and will arch in the finer cracks (0.01 in. to 0.03 in.), thus preventing complete coverage of the joint surface. Air separation is now displacing the screen for the production of fine cement for the grouting of contraction joints. The screening of cement for foundation grouting is not ordinarily necessary since poor flowage properties due to cement-particle size

can be offset by a little increased pressure. This pressure remedy, however, cannot be applied effectively to joint grouting in dams because of the danger of objectionable displacement and the creation of new and irregular cracks that would be difficult, if not impossible, to grout.

Height of Lift

It will be noted by reference to the Appendix that the height of lifts in concrete dams has varied through rather wide limits with probably more yardage placed in 5-ft lifts than in any other. The 4-ft lift was quite popular at one time in the United States, and then gradually it was displaced with the 5-ft lift, with the result that considerable economy was effected in the placing operation without much additional cost for form work. In recent years there has been considerable agitation for 10-ft lifts in massive construction, the same as in buttressed dams. This move has not proved very satisfactory, and the general result is increased cracking over that obtained in the 5-ft lift. From the construction standpoint it might be possible to perfect satisfactory and economical forms for 10-ft lifts in block or monolithic work; but from the concrete placement standpoint the results are not as satisfactory as with the thinner lifts, and, in addition, the increase in maximum temperature within the mass is decidedly objectionable.

The 5-ft lift appears to be a happy medium and when coupled with a placing schedule of five days between lifts gives reasonably satisfactory results, other things being equal. It is noted that, in the construction of Hiwassee Dam, $2\frac{1}{2}$-ft lifts with five days between pours have been used with considerable success to decrease cracking, in getting off the foundation. Douglas McHenry and Roy W. Carlson have presented[52] some pertinent and interesting data along the general line of the present paper, especially the last sentence of their concluding paragraph which notes that "It has been shown that minor refinements in the conventional design methods which determine the general shape and dimensions of a gravity dam have less effect upon stress distribution and safety than refinements in the construction procedure. This suggests that the construction program might well be considered as the governing element in the design of large gravity dams." How true! Yet how difficult it is for the constructer who is responsible for progress and economics to appreciate it.

It should be noted that of the seventy-eight dams listed in Table 11, sixty-one were completed since 1925 and only seventeen between 1889 and 1925. This ratio is fairly representative of masonry dams 100 ft or higher, it is believed. Hence, inasmuch as so little time has elapsed since quantity production of portland cement concrete dams was started, there is little wonder that the cracking is not under better control.

The outstanding dam of the seventy-eight, in point of high cement content, is the Bleiloch Dam, in Germany, completed in 1931. It should also be noted that the joints are 82 ft apart, the lifts 8 to 9 ft thick, and that cracks extended entirely through the monoliths on about 40-ft centers for the lower portion of the dam where the cement content was as high as 1.57 bbl per yd.

[52] "Measuring Dam Behavior," by Douglas McHenry and Roy W. Carlson, *Engineering News-Record*, March 30, 1939, p. 440.

The average of all transverse joint spacings that extend to the foundation for the dams listed in Table 11 is 65 ft, with a minimum of 25 ft for Deadwood Dam—a constant radius arch 165 ft high—and a maximum of 150 ft for Arrowrock Dam, in Idaho, and Pardee Dam, in California. The former is an excellent example of what can be accomplished with low cement content relative to the elimination of cracks; and the latter is equally convincing that 1 bbl of cement per yd, 5-ft lifts, and wide spacing of transverse joints on dam sites of that type will surely produce objectionable cracking.

Acknowledgments

The writer wishes to acknowledge his indebtedness to George M. Tapley, Assoc. M. Am. Soc. C. E., for assistance in assembling and compiling the material for Table 11, and to the many engineers who have made it possible to present such data.

APPENDIX

Construction Joints in Concrete Dams

The data presented in Table 11 may be further explained and, in some cases, modified, as follows:

Concrete Yardage

Referring to Col. 6, Table 11, the dams that were of cyclopean concrete and stone masonry are further described in Table 12.

Heights

Three of the dams in Table 11 (Cols. 7 and 8) were increased in height from the value given. For example, Assuan Dam (Item 4) was raised 16 ft in 1912 to a total height of 112 ft. Marshall Ford Dam (Item 49, Table 11) was completed to a height of 190 ft in March, 1939, when the Lower Colorado River Authority announced its decision to complete the high dam immediately to 265 ft. Ruby Dam (Item 66) will be raised, ultimately, to 658 ft.

Joint Spacing

Arrowrock Dam (*Item 2, Table 11*).—From the top down 70 ft the transverse joints are spaced at 25 ft; from 70 ft to 130 ft, the spacing is 50 ft; and from 130 ft to 215 ft, the spacing is 150 ft.

Ashokan (*Item 3*).—From the top of dam down 110 ft the transverse joints are spaced at 68 ft; and from 110 ft to 200 ft, the spacing is 168 ft. During construction there were transverse cracks to rock at each of the deepest expansion joints, spaced at 168 ft, with intermediate cracks about midway between.

Assuan Dam (*Item 4*).—In the Assuan Dam, a longitudinal slot 6 in. wide was left between the new masonry and the old. This slot was filled with grout after cooling. When this dam was raised (Item 5) the new concrete buttresses on the downstream face were separated from the old masonry by wrought iron, non-corrosive, bearing plates 7 mm. thick.

Barberine Dam (Item 6).—Cracks occurred near the top of Barberine Dam which were from 0.2 to 0.5 mm wide, penetrating at least 1 m from the surface.

Dams in France.—Details of the construction joints in Bissorte Dam (Item 8), Chambon Dam (Item 18), and Chavanon Dam (Item 19) are shown in Fig. 13.

Black Canyon Dam (Item 9).—At Black Canyon Dam the spacing of transverse joints was 75 ft in the spillway section and 50 ft in the abutment sections.

Bleiloch Dam (Item 10).—The Bleiloch Dam showed shrinkage cracks at about 40-ft centers, extending from the foundation to 50 ft below the top of the dam and through the entire thickness of the structure.

Bonneville Dam (Item 11).—In one monolith two vertical cracks parallel with the axis were found soon after the mass was placed. Each crack started from an abrupt change in bed rock level. The maximum opening was 0.03 in. but later decreased to somewhat less.

Boonton Dam (Item 12).—The combined width of all cracks in Boonton Dam was about 3.5 in.,[53] consisting of seventeen main cracks with top widths of $\frac{1}{16}$ in. or more; sixteen small cracks with top widths less than $\frac{1}{16}$ in.; and thirty-three half cracks part way across the top of the dam. Main cracks occur at very regular intervals of 100 ft and no effort has been made to treat any of these cracks or reduce the leakage.

Boulder Dam (Item 13).—In Boulder Dam the transverse joints are spaced 60 ft on the axis and vary from a minimum of 25 ft at the downstream toe to a maximum of 66 ft at the upstream face of the dam. The circumferential joints vary from 30 ft near the downstream toe to 50 ft at the upstream toe.

Camarasa Dam (Item 16).—From the crest of Camarasa Dam downward 72 ft, the transverse spacing of joints was 52.5 ft, as shown in Table 11, Cols. 10 and 11.

Canning Dam (Item 17).—The spacing on Canning Dam is 45 ft from the crest down to 80 ft below the crest. For the lower part of this dam, the spacing is 90 ft.

Claytor Dam (Item 22).—In the lower part of a few large monoliths of Claytor Dam there are longitudinal joints about midway between faces. These joints are not grouted; but the transverse contraction joints have an 18-in. strip of expansion joint filler at both upstream and downstream faces. In each horizontal 5-ft lift joint there is a 24-in. by 6-in. raised key, a copper-bearing steel plate water stop and 8-in. by 10-in. drains 3 ft, 5.5 ft, and 7.5 ft, respectively, from the upstream face. To prevent the retention of any inferior material at the top of each lift, a strip 18 in. wide and 2 in. to 5 in. thick (depending on the slump) was removed before the initial set occurred, along all permanently exposed faces. Joints in the intake, the bulkhead, and the spillway sections are spaced on 44-ft, 54-ft, and 60-ft centers.

Cignana Dam (Item 23).—The principal joints of Cignana Dam, at 98-ft centers, intersect the entire cross section from crest to foundations. Secondary joints occur from the upstream face to the inspection pit (see Fig. 9).

[53] "The Effect of Temperature Changes on Masonry," by the late Charles S. Gowen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXI, December, 1908, pp. 391–409, 410, and 413.

TABLE 11.—COMPARATIVE DATA ON

Dam No.	Name	River	Location	Year finished	Type[a]	Volume, in thousands of cubic yards	Dimensions, in Feet: Height Above: Foundations	Low water	Length
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Ariel	Lewis	Washington	1931	CA	200	313	192	1,300
2	Arrowrock	Boise	Idaho	1916	CG	585	351	....	1,100
3	Ashokan	Esopus	New York	1911	SG	500[h]	252	210	1,000
4	Assuan	Nile	Egypt	1912[e]	SG	704[h]	96[h]	....	6,400
5	Assuan (Raised)	Nile	Egypt	1934	SG	1,730	142	....	6,970
6	Barberine	Barberine	Switzerland	1925	CG	288[h]	259	174	930
7	Big Tujunga	Big Tujunga	California	1931	CA	108	249	....	585
8	Bissorte	Bissorte	France	1935	G	392	196	....	1,780
9	Black Canyon	Payette	Idaho	1924	G	79	184	....	1,134
10	Bleiloch	Saale	Germany	1931	CG	236	213	....	700±
11	Bonneville	Columbia	Ore.-Wash.	1938	SG	500	170	90±	1,250
12	Boonton	Rockaway	New Jersey	1905	G	225[h]	114	110	2,150
13	Boulder	Colorado	Nev.-Ariz.	1935	CG	3,250	726	....	1,282
14	Bull Run	Bull Run	Oregon	1929	CG	222	200	....	1,000
15	Calderwood	Little Tennessee	Tennessee	1930	CA	280	232	200	916
16	Camarasa	Pallaresa	Spain	1920	CG	283	333	....	460
17	Canning	....	Australia	1936	G	....	218	....	1,600
18	Chambon	Romanche	France	1935	SG	392	450	290	1,080
19	Chavanon	Chavanon	France	1925	CG	445	295	....	1,180
20	Cheoah	Little Tennessee	N. Carolina	1919	....	200	225	200	750
21	Chute-a-Caron	Saguenay	Quebec	1931	SG	460	200	170	3,040
22	Claytor	New	Virginia	1939	SG	217	132	125	1,140
23	Cignana	....	Italy	1928	CG	200	183	....	1,320
24	Conchas	South Canadian	New Mexico	1939	SG	750	235	190	1,250
25	Cross River	Cross	New York	1908	SG	161[h]	170	126	987
26	Croton Falls	West Croton	New York	1911	G	260[h]	173	113	1,100
27	Cushman No. 1	Skokomish	Washington	1925	CA	90	275	250	786
28	Cushman No. 2	Skokomish	Washington	1931	CR	....	240	170	580
29	Deadwood	Deadwood	Idaho	1931	CR	55	165	....	750
30	Diablo	Skagit	Washington	1928	CA	330	389	....	1,180
31	Dniepostroy	Dnieper	Russia	1932	G	950	198	153	2,500
32	Don Pedro	....	California	1923	CG	282	288	278	1,040
33	Elephant Butte	Rio Grande	New Mexico	1916	SG	605[h]	306	....	1,155
34	Eguzon	Creuse	France	1927	CG	274	200	....	780
35	Exchequer	Merced	California	1926	CG	369	330	309	955
36	Fifteen Mile Falls	Connecticut	Vermont	1930	SG	....	175	....	....
37	Friant	San Joaquin	California	1943	SG	1,900	300	255	3,430
38	Gibson	Sun	Montana	1930	CR	161	205	....	960
39	Gilboa	Schoharie	New York	1926	SG	....[h]	182	155	1,324
40	Grand Coulee	Columbia	Washington	1940	SG	10,000	540	368	4,140
41	Grimsel	Aar	Switzerland	1931	CG	445	377	328	820
42	Hiwassee	Hiwassee	N. Carolina	1940	SG	770	307.5	....	1,265
43	Hogan	Calaveras	California	1930	CA	122	136.5	113	1,325
44	Hohenwarte	Saale	Germany	1939	CG	608	246	138	....
45	Kensico	....	New York	1916	SG	....[h]	307	168	1,843
46	Laggan	Spean	Scotland	1934	CG	110	186	....	680
47	Madden	Chagres	Panama	1934	SG	525	223	....	974
48	Mareges	Dordogne	France	1935	CA	242	295	....	650
49	Marshall Ford	Colorado	Texas	1939	SG	969	265[h]	....	2,325
50	Martin	Tallapoosa	Alabama	1926	CG	395	168	157	1,300
51	Melones	Stanislaus	California	1926	CR	93	210	....	590
52	Mettur	Canvery	India	1934	SG	2,030	230	....	7,707
53	Montejague	Guadares	Spain	1925	CA	35	273	230	259
54	Morris	San Gabriel	California	1934	CG	450	328	245	756
55	Mount Bold	Onkaparinga	Australia	1937	CR	132	141	130	700
56	Narrows	Yadkin	N. Carolina	1917	CG	525	216	185	1,144
57	New Croton	Croton	New York	1906	SG	855[h]	297	174	1,168
58	Norris	Clinch	Tennessee	1935	SG	1,000	265	....	1,570
59	O'Shaughnessy	Tuolumne	California	1923	CG	398	344.5	226.5	605
60	O'Shaughnessy (Raised)	Tuolumne	California	1938	CG	675	430	312	900
61	Owyhee	Owyhee	Oregon	1932	CG	504	405	....	850
62	Pacoima	Pacoima	California	1927	CA	226	380	355	600
63	Pardee	Mokolumne	California	1929	CG	640	358	350	1,320

Construction Joints in Concrete Dams

Joint Spacing, in Feet		Height of lifts, in feet	Hours between pouring lifts	Barrels of Cement per Cubic Yard of Concrete[b]		Water-cement ratio, by weight	Dimensions of Keys, in Inches	
Transverse	Longitudinal			Interior	Exterior		Size	Spacing, on centers
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
30±	....	10	....	1.0	....	0.71	54 × 6[d]	...
25, 50, 150	....[h]	4	....	0.96	1.05[b1,h]	....	36 × 10[f]	120
84, 168	....[h]	....	....	1.00	1.30[b2]	....	120 × 12[d]	...
None	....[h]	....	....	....	....[b2]	....	....	...
23	....[h]	....	....	....	....[b2]	....	....[f]	...
82[h]	....	5±	72–120	0.9	....[b2,h]	0.75–0.80	None	...
50	....	....	72	1.05	....	0.73	....	...
49[h]	....	....	....	....	....	....	Broad triangular keys	...
75, 50[h]	....	3.5	72	0.88	....[b2]	....	48 × 9[d]	...
82[h]	....	8 to 9	120–144	1.15 to 1.57[h]	....[b3]	0.69	....	...
60	....	5	72	0.9 to 1.01[h]	1.21 to 1.45[b4]	0.52–0.75	....	...
None[h]	....	....	....	0.68	....[b2]	very wet	....	...
25 to 66	30 to 50[h]	5	72	1.01	....[b3]	0.59	30 × 6[d]	...
40	....	4.5	48	0.81	0.96	0.71	20 × 8[f]	...
50±	....	10	....	1.12	....[b2]	0.67	30 × 8[f]	78
52.5[h]	....	....	....	1.22	....[b1,h]	....	....[f]	...
45, 90[h]	....	6	....	1.10±	1.40±[h]	0.59–0.63	....[f]	...
49[h]	....	....	....	0.67 to 1.0	1.12	0.73	240 × 16±[d]	...
49	....	....	....	....	....	....	240 × 16±[f]	...
50	....	5	....	1.1±	....[b2]	....	30 × 8[f]	78
50 to 55	....	10	....	0.78	1.16[b2]	0.8–0.6	48 × 6[f]	120
44, 54, 60	45[h]	5	72	0.85 to 1.02	120 to 1.35[b5,h]	75, 57[e]	30 × 6[f,g]	120
28, 49, 98	....[h]	....	....	....	....	....	....	...
40 to 50[h]	30 to 50[h]	5	72	0.75 to 0.94	1.00 to 1.19[b5]	0.69–0.62	54 × 12[f]	216
None[h]	....	....	12[h]	0.75	....[b2]	wet	....	...
None[h]	....	....	....	0.65	1.25±[b2]	....	....	...
75	....	5[h]	72–96	1.20±	....[b2]	Slump 4½ in.	....[f]	...
36 to 52[h]	....	5	48	1.25	....[b2]	0.53	96 × 12[d]	...
25	....	4	72	1.32	....[b2]	0.53	36 × 9[f]	...
75	....	5	....	1.27	1.46[h]	0.47	72 × 12[d]	...
50±	40	13	....	....	....	....	....[f]	...
32.5, 65, 130	....[h]	4[h]	24	0.82	....	....	60 × 12[f]	...
100, 50	....[h]	4	....	1.00	1.40[b1,h]	0.65–0.80	18 × 4[f]	...
98	....	....	....	....	....	....	78 × 12[f]	...
25, 50, 75	....	5	....	0.79	richer	0.71–0.8	60 × 12[f]	...
30 to 50[h]	....	....	....	....	....[b2]	....	....	...
50	....	5	72	0.8[h]	1.0[b3]	0.6	30 × 6[d]	...
30, 60[h]	....	4	72	1.0	....[b2,h]	0.67	36 × 9[d]	...
76[h]	....	3 to 6	8–24	1.4	....[b2]	0.41–0.63	....[f]	...
50	50	5	72	1.0	....[b6,h]	0.60	30 × 6[d]	...
25, 49, 98	50[h]	....	....	0.85	1.34	0.83	....	...
38 to 50[h]	....	5[h]	60	0.85	1.2[b3]	0.57	....[f]	...
30 to 40	....	5	....	0.94	....[b2]	0.63	42 × 12[d]	...
49	....	5.9	....	1.28	....[b3,h]	0.67	....[f]	...
73.5 to 79	....[h]	....	....	0.837	....	....	....[f]	...
45	....	3.5	....	1.12	2.0[b2,h]	0.4–0.55	....	...
56	....	5	72	1.00	....[b2]	0.70	36 × 9[d]	...
43	....	....	....	1.13	1.22 to 1.34	0.45	....[f]	...
52	42	5	....	....	....[b3]	....	30 × 6[d]	...
30 to 72	....	10	24	1.00±	....[b7,h]	....	....[f]	...
....	....	4	....	0.78	....	0.75	....	...
126	....	....	....	....[h]	....[b2,h]	....	....	...
40 to 80	....	4	120	0.88	....[b2]	....	....	...
50	....	5[h]	....	0.95	1.10[b3]	0.59–0.54	....	...
40[h]	....	....	....	1.10±	....	0.63–0.67	....[f]	...
50	....	5	....	1.25±	....[b2]	....	30 × 8[f]	78
None[h]	....	....	....	....	....[b2]	....	....	...
56[h]	....	5[h]	72	0.90	1.10 to 1.20[b5,h]	0.65–0.57	....	...
97	....	5	 72	1.00	1.25[b2]	....	180[f]	...
48.5	....[h]	5		1.00	1.19[b2]	0.62–0.58	180[f]	...
50	....	4	72	1.0	1.25[b2]	0.67–0.57	36 × 9[d]	...
50	....	5	....	1.04	....	0.73	....	...
37.5, 75, 150[h]	....[h]	5	....	1.00	....	0.67	....[d]	...

TABLE 11.—

Dam No.	Name	River	Location	Year finished	Type[a]	Volume, in thousands of cubic yards	Dimensions, in Feet: Height Above: Foundations	Height Above: Low water	Length
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
64	Parker..........	Colorado	Calif.-Ariz.	1939	CR	260	320	....	800
65	Roznow.........	Dunajee	Poland	1939	SG	500	160	....	1,800
66	Ruby...........	Skagit	Washington	1940	CA	....	230[h]	140	490
67	St. Francis......	San Francisquito	California	1926	CG	175	205	....	650
68	San Mateo......	....	California	1889	CG	157.2	154	140	680
69	Santeetlah.......	Cheoah	N. Carolina	1928	CA	195	214	187	1,120
70	Sarrans.........	....	France	1930	G	....	377	....	710
71	Sautet..........	Drac	France	1934	CA	130	414	....	263
72	Schraeh.........	Waeggital	Switzerland	1925	G	305	362	220	550
73	Seminoe.........	North Platte	Wyoming	1939	CA	161	290	....	540
74	Shasta..........	Sacramento	California	1943	CG	5,400	560	528	2,860
75	Shoshone........	Shoshone	Wyoming	1910	CR	79[h]	238	....	200
76	Tygart..........	Tygart	W. Virginia	1937	SG	1,200	240	....	1,850
77	Waterville.......	Big Pigeon	N. Carolina	1929	CA	123	200	190	790
78	Wilson..........	Tennessee	Alabama	1926	SG	1,240	137	...	4,860

[a] Symbols denote: CA, constant angle; CG, curved gravity; SG, straight gravity; G, gravity; CR,
[1] Sand-cement; [2] portland; [3] low-heat; [4] portland pozzuolona; [5] moderately low heat; [6] moderate and low
[g] 12 on 1, side batter. [h] See modifying comments in the accompanying text.

Conchas Dam (*Item 24*).—The transverse spacing of 40 ft in the spillway section of Conchas Dam was dictated by the spacing of the sluicing conduits. The joints in the non-overflow section are from 40 ft to 50 ft, on centers. Longitudinal joints were provided in the abutment monoliths only, which were placed in blocks in order to maintain the lateral bracing of the abutments during construction. These joints were grouted.

Cross River Dam (*Item 25*).—During the construction of Cross River Dam, four major cracks were observed. They had a combined total width of $\frac{7}{8}$ in. at the top of the dam and extended 42 ft to 70 ft down the face. Leakage indicated a crack depth of 43 ft, the distance between cracks being 62 ft, 165 ft, and 226 ft.

Croton Falls Dam (*Item 26*).—In the second winter after the construction of Croton Falls Dam three major and several minor cracks appeared. There is considerable seepage every winter, which disappears in summer.

Cushman Dam No. 1 (*Item 27*).—Many horizontal joints showed a slight seepage at first, but sealed themselves in time, leaving the back of the dam covered with calcium deposit.

Cushman Dam No. 2 (*Item 28*).—Some cracking and seepage encountered in Cushman Dam No. 1 were eliminated except for two diagonal cracks at a sharp break in the profile of the foundation and in the base of the left abutment, and one horizontal crack in the thrust block near the top of the right abutment.

Don Pedro Dam (*Item 32*).—From the top of Don Pedro Dam down 62 ft, the transverse joints are spaced at 32.5 ft; from 62 ft down to 127 ft, at 65 ft; and in the remaining height, 130 ft.

(Continued)

Joint Spacing, in Feet		Height of lifts, in feet	Hours between pouring lifts	Barrels of Cement per Cubic Yard of Concrete[b]		Water-cement ratio, by weight	Dimensions of Keys, in Inches	
Transverse	Longitudinal			Interior	Exterior		Size	Spacing, on centers
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)
50	....	5	72	1.00	....[b3]	0.6	30 × 6[d]	...
49	....	....	....	1.12	1.34[b2]	....	....	...
50	....	5	72	1.00	....[b3]	....	60×60×12[d]	...
None[h]	....	....	....	1.12	....[b2]	....	....	...
Irregular blocks	....	....	....	1.23	....[b8,h]	0.44	....	...
50, 40, 37[h]	....	5, 10	....	1.25[h]	1.35[b2,h]	0.64–0.70	30 × 8[f]	78
52	....	....	....	1.0 to 1.3[h]	....[b2]	....	....	...
85±	....	....	....	0.8 to 0.98	0.98 to 1.22	....	....	...
....	....	....	....	0.85 to 1.0	1.34[h]	0.94–0.63	....	...
50	....	5	72	....	....[b5]	....	30 × 6[d]	...
50	50	5	72	1.0	....[b3]	0.6	30 × 6[d]	...
None	....	....	....	1.0±	....	....	....	...
52, 60	....	5, 8, and 10	143, 206, and 212	0.75 to 1.0	1.25[b6]	0.48–0.65	48 × 12[d]	...
50	....	5, 10	....	1.20 [e]	....	....	....[f]	...
38, 48, 54[h]	....	4 to 6	....	1.37	1.62[h]	....	24 × 10[f]	...

constant radius. [b] The types of cement, for interior as well as exterior concrete, are identified as follows: heat; [7] slag; and [8] coarse. [c] Interior, 75; exterior, 57. [d] Keyway. [e] First stage completed in 1902. [f] Keys.

Elephant Butte Dam (Item 33).—The spacing of the transverse joints of Elephant Butte Dam is 50 ft for the top half and 100 ft for the bottom half.

Fifteen Mile Falls (Item 36).—At the end of the first year following the completion of Fifteen Mile Falls Dam there were six of the eight 45-ft spillway monoliths, in which intermediate cracks had developed, and only four of the ten 32-ft monoliths that indicated any cracking. A spacing of somewhat less

TABLE 12.—Dams of Cyclopean Concrete and Stone Masonry

Dam No.	Name	Plum stone (percentages)	Concrete mix	Facing
3	Ashokan	25	......	Precast concrete blocks
4	Assuan	(Granite masonry with portland cement mortar)		....
6	Barberine	7 to 10	......	Ashlar
12	Boonton	50±	1 : 2.75 : 6.25	Ashlar[c]
25	Cross River	35	......	Concrete blocks
26	Croton Falls	32 to 40	1 : 3 : 6	Concrete blocks
33	Elephant Butte	15±	1 : 2.8 : 5.45	....
39	Gilboa	5.8±	......	Stone masonry
45	Kensico	27	......	....
50	Martin	15 to 20[a]	1 : 3 : 6	Stone masonry[d]
52	Mettur	(Plain concrete; upstream face of rubble masonry, in a specially rich mortar—1 : 2.75)		
57	New Croton	(Rubble masonry, faced with ashlar)		....
75	Shoshone	........[b]	1 : 2.5 : 5	....

[a] Cobbles and plum stones. [b] Clean, sound pieces of granite, hand placed in concrete. [c] Slight leakage when dam was subjected to its full head. [d] Concrete blocks on upstream face and stone masonry on downstream face.

than 30 ft would possibly have eliminated all cracking. All intermediate cracks start from the foundation and disappear before they reach the top of the dam.

Gibson Dam (*Item 38*).—Gibson Dam has its transverse joints spaced at 30 ft for the top 75 ft of the dam and 60 ft for the bottom part.

Gilboa Dam (*Item 39*).—At Gilboa Dam cracks in two of the monoliths have caused some leakage. However, no consistent cracking has occurred between joints, although the monoliths are 72 ft long.

Grimsel Dam (*Item 41*).—Longitudinal cracks, $\frac{1}{16}$ in. wide and 40 to 60 ft apart, occurred in the base of Grimsel Dam. These cracks were covered with a heavy reinforcing steel mat, and grouted. In subsequent construction longitudinal joints were provided.

Hiwassee Dam (*Item 42*).—The spillway of Hiwassee Dam occupies 260 ft of the central part of the structure. The spacing of joints in this section is 38 ft, with one monolith at the left end 40 ft wide. The joints on both abutments are 50 ft on centers. No cracks have appeared on the faces of the dam and only a few longitudinal cracks between transverse joints.

Kensico Dam (*Item 45*).—At Kensico Dam superficial cracks appeared across the top of the dam between monolith joints. There cracks could not be found in the inspection galleries 6 ft below.

Mount Bold Dam (*Item 55*).—Shrinkage stresses were reported to be rare in Mount Bold Dam. When they occurred they were radial, usually on the upstream face, and then only when the concrete was placed in hot weather.

New Croton Dam (*Item 57*).—In March, 1906, when the New Croton Dam was substantially completed, there were four cracks $\frac{1}{8}$ in. wide and one crack $\frac{1}{4}$ in. wide, as measured in the tunnel 6 ft to 9 ft below the top. Smaller cracks that appeared at coping joints were not visible in the tunnel. Seepage was first observed at two points during the winter of 1938–1939 which disappeared the following summer.

Norris Dam (*Item 58*).—At Norris Dam the spacing of joints is 56 ft throughout, except for the power-house headworks in which there are two monoliths 60 ft wide, one 62.5 ft wide, and one 40 ft wide.

O'Shaughnessy Dam (*Raised*) (*Item 60*).—When O'Shaughnessy Dam was raised, a slot 3.5 ft wide was included to separate the new concrete from the old.

Pardee Dam (*Item 63*).—From the top of Pardee Dam down 79 ft, the transverse joints are spaced at 37.5 ft; from 79 ft to 180 ft down, at 75 ft; and from 180 ft to the foundation, at 150 ft.

St. Francis Dam (*Item 67*).—Contraction cracks at St. Francis Dam occurred in approximately radial planes at 50 ft to 75 ft spacing; but it was concluded that these were not of such a nature as to affect the stability of the structure. The dam failed on March 12, 1928.

Santeetlah Dam (*Item 69*).—The spacing shown in Col. 10, Table 11, for this dam applies to the bulkhead, arch, and spillway, respectively.

Wilson Dam (*Item 78*).—The spacings of 38 ft and 48 ft for the spillway structure were dictated by the bridge pier location; 54 ft for the headworks structure was determined from the spacing of the hydroelectric units.

Cement

Table 11 gives 12 hr as the time between pouring separate lifts on Cross River Dam (Item 25). Actually the permissible minimum range was 12 to

16 hr, with a permissible average vertical rise of 18 ft per month. The values shown for Don Pedro Dam (Item 32) were further limited to 40 ft in 30 days. In the Hiwassee Dam (Item 42), 2.5-ft lifts were used on all rock foundations and on concrete that was more than 15 days old. On 15-day concrete, two, and on 30-day concrete, four, 2.5-ft lifts, were required before any single 5-ft lift. The 5-ft lifts of Morris Dam (Item 54) and Norris Dam (Item 58) slope upward 10% and 5%, respectively, in a downstream direction.

Type of Cement.—Special types of cement that could not be described adequately in Table 11 are as follows:

Dam No.	Discussion
2	Portland cement reground with finely ground granite in proportions of 55% cement and 45% granite. Fineness was gaged by a permissible 10% retained on a 200-mesh sieve.
6	Manufactured in revolving tubular kiln, under Swiss specifications.
10	Mix: 34 parts of portland and 66 parts of an admixture, by weight. As indicated in Col. 14, the cement concentration varied from 1.15 bbl at the top of the dam, 1.35 bbl at the mid-section, and 1.57 bbl per cu yd at the base. Note the cracking due to the high cement content in the lower part of the dam.
16	Portland cement reground with crushed sandstone in the proportions of 56% cement and 44% sandstone; fineness standard same as item 2.
22	Both fine and coarse aggregate manufactured from dolomite. Approximately 10% hydraulic lime was used in the interior. The cement content was reduced gradually, because of cracking.
30	For a thickness of 3 ft, along the upstream face of the dam, the cement concentration was 1.46 bbl, as stated in Table 11.
33	Portland cement was reground with crushed sandstone in the proportions of 52% cement and 48% sandstone; fineness standard same as item 2. As shown in Table 11, a somewhat richer mix was used in the upstream face only. The upstream face was also coated with gunite.
37	In addition to the 0.8 bbl of cement, the interior of this dam also contained about 60 lb (1.5 cu ft) of pumicite per cubic yard.
38	To the portland cement an admixture of 1% to 2% diatomaceous earth is added to correct the harshness in the sand when necessary.
40	Although cracking at Grand Coulee Dam was greatly reduced as compared with Owyhee (Item 61) the conditions were not all that might be desired. The decision was made to use low-heat cement to complete the dam. Analysis of crack surveys at three dams of the Bureau of Reclamation

shows the following results in length (feet) of opened cracks per foot of galleries:

Dam No.	Name	Unit length
61	Owyhee	2.47
40	Grand Coulee	1.85
13	Boulder	0.72

44 Portland cement and trass (volcanic tufa) finely pulverized, in the proportions of 60% cement and 40% trass, which delays the setting of concrete and also unites with the lime in cement to form calcium silicate. A patented admixture was used.

46 Fineness standard, 7% retained on a 170-mesh sieve. Referring to Table 11, 2.0 bbl of cement per cubic yard were used in the upstream face of the dam only. Shrinkage cracks were observed on all faces of monoliths, at about 15-in. centers.

50 85% portland and 16% slag cement.

52 80% portland and 20% underburned pulverized brick.

58 In one monolith a slag-cement blend was tried—75% portland and 25% slag cement. Strength and temperature were not changed materially, and shrinkage cracks were still formed.

68 An imported cement, extremely coarse in terms of present-day practice; 10% retained on a sieve having 50 meshes per lin in. The concrete was very dry and was hand-tamped in 3-in. layers. After 50 years of service this dam was found to be in excellent condition.

69 In the gravity section, 125 bbl per cu yd and in the arch section 1.35 bbl.

70 The foundation was of portland cement, 1.0–1.3 bbl per cu yd; and slag cement was used for the upper portion.

71 Two slots, each 4 ft wide, extended through the dam from the upstream to the downstream face, and from the crest to 316 ft below the crest.

72 For the upstream face and in the top 50 ft of the dam, 1.34 bbl per cu yd were used, as shown in Table 11. The downstream face, containing less than 1.0 bbl per cu yd, disintegrated badly and was covered, in 1930, with a stone facing. At an altitude of 3,000 ft, the weather conditions at the site are severe.

78 For all the reinforced concrete, and for the concrete in the dam above the rollway crest, the cement used was 1.62 bbl per cu yd.

Methods of Cooling

A summary of problems concerned with the cooling of concrete is as follows:

Dam No.	Discussion
1	Slots 2 ft wide were left between monoliths, for cooling. River water which was circulated through vertical pipes placed in several monoliths was found to be very effective in reducing temperature. The slots were filled six months after completion.
2	At each joint there was one 5-ft square shaft and two 10-ft square shafts. These wells were filled with concrete during cold weather.
3	Specifications provided one 5-ft by 3-ft well in each joint to be filled with some elastic watertight substance if needed to stop leakage.
4	Longitudinal slots, 6 in. wide, were left between the new masonry and the old. They were filled with grout after cooling.
6	Grouted six months after completion.
7	Provision was made for grouting.
11	Provision was made for grouting.
13	The concrete was first air cooled; later refrigerated water was circulated through 1-in. tubing on 5-ft centers embedded in each lift. After cooling, all joints were grouted in 50-ft lifts.
15	Grouted after completion.
22	Provision was made for grouting the transverse joints only. No provision was made for cooling.
24	Longitudinal joints were provided in abutment monoliths only, the monoliths being placed in blocks in order to maintain the lateral bracing of the abutments during construction. These joints were grouted after cooling.
27	Grout system was never used.
28	Construction details for Cushman Dam No. 2 are shown in Fig. 12. Grout system was never used.
29	Grouted six months after completion.
30	Provision was made for grouting.
37	River water circulating through 1-in. tubing embedded in each lift at 5.0-ft centers near the top of the dam and 2.5-ft centers near the base. Provision was also made for pre-cooling the materials during the summer months.
38	Grouted the winter after completion.
40	River water was circulated through 1-in. metal tubing embedded in each lift. This cooling system was grouted as the construction proceeded.

41 The slats were filled eighteen months after completion.

42 Cooled by refrigerated mixing water; also river water was circulated through 1-in. tubing placed in each lift at points where excessive temperatures were anticipated. In this dam, keys were placed parallel to the downward slope of the dam.

43 Grouted eighteen months after completion.

44 During cold weather river water was circulated through 1.5-in. tubing on 4.92-ft centers, embedded in each lift. During hot weather, refrigerated circulating water was used. No cracks had been observed in the dam to the time of its completion. This cooling system was grouted after serving its purpose.

47 This dam was completed in June, 1934. Reservoir water was pumped through the grouting system during the cool months, until in September, 1936, it was grouted.

49 Provision was made for grouting.

55 Joints were grouted with cement mortar under pressure, after maximum cooling and shrinkage.

59 Joints were grouted in 1938.

60 River water and refrigerated water were circulated through 1-in. tubing spaced 5.5 ft, embedded in each lift. After the new concrete had cooled to 45° F the slot was filled with concrete. When this cooled, the joint between the new mass concrete and the slot concrete was grouted.

61 River water was pumped through the contraction joint grouting system. The joints were then grouted in the spring after the dam was completed. Grouting was done during cold weather.

63 The cooling system was grouted in cold weather. It was grouted a second time in 1932 to reduce seepage and plans were made to grout it a third time if necessary. Some shrinkage cracks, large enough to permit seepage, have occurred.

64 River water, only, was circulated through 1-in. metal tubing embedded in each 5-ft lift, the minimum velocity being 2 ft per sec. This system was grouted after cooling.

66 Provision was made for grouting.

68 Keyed on all sides of each block.

69 Grouted immediately after construction.

71 Two slots, each 4 ft wide, extended through the dam from the upstream to the downstream face and from the crest to 316 ft below the crest. They were filled after cooling.

73 River water was circulated through 1-in. tubing on 5.5-ft centers embedded in each lift. The minimum velocity of flow was 2 ft per sec, and water temperatures varied from

	32° to 56° F. The maximum concrete temperature was estimated at 90° F. Grouted after cooling.
74	River water was circulated through 1-in. tubing on 5.75-ft centers, embedded in each lift. Temperatures of 50° to 90° F are reduced so that volume changes can be completed in time to permit joint grouting as construction proceeds.
76	The bottom 20 ft of each joint were grouted ten months after completion.
77	Slots, 8 ft wide, were filled during cold weather.

Copper Water Stops

The material used for water spouts was predominantly copper, 31 dams reporting the use of this material. The following deserve special comment:

Dam No.	Discussion
7	In Big Tujunga Dam, copper stops were located 12 in. from the upstream face, with galvanized iron grout stops near the downstream face.
14	In Bull Run Dam, 24-in. copper stops were shaped to cover the downstream half of a vertical well, 6 in. in diameter, filled with moist well-tamped clay.
17	Canning Dam had a copper water stop and an asphalt well 8 in. in diameter.
19	In Chavanon Dam there was a flat copper strip upstream from a circular drainage well.
22	In each transverse contraction joint of Claytor Dam, 5.5 ft from the upstream face, there is a V-type stop with 8-in. legs welded to horizontal steel water stops at each 5-ft lift.
24	A 24-in. copper stop in Conchas Dam is shaped to cover the upstream half of a vertical well, 6 in. in diameter, filled with asphalt.
34	In Eguzon Dam, the stops are upstream from a 3-ft square drainage well and,
35	at Exchequer Dam, the stops are upstream from a 10-in. square drainage well.
43	Hogan Dam, similar to Big Tujunga, had V-shaped copper stops upstream and galvanized iron grout stops downstream. The Troiel grout system was used, with 2-in. half round metal forms in the joints.
44	In Hohenwarte Dam a vertical inspection shaft extends the entire height of the dam, in each monolith joint. Downstream from the shafts are vertical and horizontal drain pipes.
45	In Kensico Dam, in addition to copper stops there is also a 4-ft by 5-ft inspection shaft in each joint.
51	Melones Dam has a 10-in. drainage well and copper seal.

54	Morris Dam has copper water seals in 8-in. wells, 2.33 ft from the upstream face.
55	Mount Bold Dam has copper at the upstream face and galvanized iron at the downstream face.
61	At Owyhee Dam there are V-shaped, 20-gage, soft copper stops at both faces, around all gallery openings, and at each 100-ft level.
64	The stops for Parker Dam are flat copper, coated with an asphalt emulsion $\frac{1}{16}$ in. thick.
66	Ruby Dam has V-shaped copper water stops near both upstream and downstream faces.
77	In Waterville Dam there are copper verticals and sheet iron horizontal water stops in each lift.

Other structures reporting copper water stops may be cited without further comment, as follows: Ariel (Z-section), Arrowrock (Z-section), Ashokan, Bissorte (Z-section), Black Canyon, Chambon (Z-section), Deadwood, Don Pedro, Friant, Gibson, Gilboa, Grand Coulee, Madden (V-section), O'Shaughnessy (Dam 59) and Seminoe.

Asphalt Water Stops

The material next in general importance to copper was asphalt, which was reported from ten dams listed in Table 11. These merit special comment as follows:

Dam No.	Discussion
5	The raised section of Assuan Dam was fitted with $\frac{1}{2}$-in. steam pipe for melting the asphalt.
27	Cushman Dam No. 1 has asphalt stops upstream and copper downstream. The steam pipe provided for heating the asphalt has never been used. All horizontal pipes (see Fig. 12) and all vertical pipes (four) have $\frac{1}{4}$-in. by 12-in. slots on both sides of the steam pipe in line with the contraction joint. The slots are 2.0 ft, on centers, and are covered with paper and cheese cloth.
28	Cushman Dam No. 2; same as item 27.
30	In Diablo Dam there is an asphalt stop with a V-shaped copper strip 1 ft wide. A steam pipe was provided for heating the asphalt.
31	In Dnieprostroy Dam the stops are square wells filled with asphalt.
46	Laggan Dam—wells, 6 in. in diameter, filled with asphalt and equipped with copper seals.
52	Mettur Dam—a 12-in. square concrete staunching post coated with asphalt.
26	Pacoima Dam—asphalt wells near the upstream face and copper stops near the downstream face.

69	Santeetlah Dam—6-in. square wells filled with asphalt.
78	At Wilson Dam, joints were filled with tar paper. After leakage developed wells were drilled in each joint and filled with asphalt. The leakage, however, has not stopped completely.

Miscellaneous Types of Water Stops

The original Assuan Dam had no water stops. Hiwassee Dam used stainless steel. Other variants from the general rule are as follows:

Material	Remarks
Monel	Boulder Dam has monel water stops and copper grout stops near the upstream face. Galvanized iron grout stops were placed near the downstream face and in all intermediate joints. Tygart Dam has monel **Z**-sections near both faces.
Steel	Calderwood Dam has **V**-shaped, 26-gage steel grout stops in both faces.
Lead	Cheoah Dam and Narrows Dam have **V**-shaped lead strips and an asphalt wedge in each construction joint.
Drains	For water stops, Chute-a-Caron Dam has one well 6 in. square in each joint. Elephant Butte Dam has two open wells, 6 in. in diameter, in each joint.
Jute	Water stops in the Cignana Dam are characterized by an elastic layer of jute and bitumen, protected by a butt joint in reinforced concrete, embedded in the upstream face of the dam.
Galvanized iron	Pardee Dam and the raised section of O'Shaughnessy Dam have galvanized iron grout stops in both faces.
Concrete	In Sarrans Dam there is a reinforced concrete needle at the upstream face and across each monolith joint.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DESIGN OF DOWELS IN TRANSVERSE JOINTS OF CONCRETE PAVEMENTS

Discussion

By Bengt F. Friberg, Assoc. M. Am. Soc. C. E.

Bengt F. Friberg,[21] Assoc. M. Am. Soc. C. E. (by letter).[21a]—The purpose of the paper was to present the rules governing the behavior of an elastic structure in a yielding mass in such form that their application to steel dowels, encased in concrete and crossing transverse joints in concrete pavements, would be convenient to the designing engineer. To cover the subject the paper included consideration of dowel spacing as well, although it must be recognized that any analytical evaluation of dowel spacing upon pavement stresses is subject to considerable approximation.

Credit for first having indicated the application of bent bars in an elastic foundation to the design of single dowels in concrete pavement joints may be claimed by Dean Grinter.[2] The general solution by Messrs. Timoshenko and Lessels[4] represents the actual conditions around a dowel closer than most other applications; it has been used advantageously in the far less exact structure of railroad tracks. Such approximations as occur are the results rather of non-uniformity in the elastic behavior of the surrounding concrete as evaluated by the modulus of support "K" than in approximations of the solution.

In developing the formulas for dowel design, a number of assumptions were made. Mr. Fremont has enumerated, checked, and substantiated several of these assumptions, and, based on his discussion, the following conditions presented in the paper may be agreed upon as approaching the actual conditions nearly enough for present-day designs:

(*a*) Hooke's law for bearing pressure as proportionate to deflection at any point along a dowel may be applied when contact between the dowel and the concrete exists;

Note.—This paper by Bengt F. Friberg, Assoc. M. Am. Soc. C. E., was published in November, 1938, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1939, by L. E. Grinter, M. Am. Soc. C. E.; and June, 1939, by W. O. Fremont, M. Am. Soc. C. E.

[21] Research Engr., Laclede Steel Co., St. Louis, Mo.

[21a] Received by the Secretary March 27, 1940.

[2] "Design of the Reinforced Concrete Road Slab," by L. E. Grinter, M. Am. Soc. C. E., *Bulletin No. 39*, Texas Eng. Experiment Station.

[4] "Applied Elasticity," by S. Timoshenko and J. M. Lessels, Westinghouse Technical Night School Press, pp. 133–141.

(*b*) Flexible steel dowels of round bars may be assumed to act as hinges between the two slabs with a point of contra-flexure at the center of the joint;

(*c*) Critically high bearing stresses, although sufficient to cause plastic deformation or failure in the concrete around the dowel, need not be considered critical for the pavement;

(*d*) The dowel length in contact with the concrete need not be greater than to the second stress-change point on each side of the joint;

(*e*) Pavement slope at the joint can be neglected in computing the deflection of a dowel across the joint; and

(*f*) Misalignment stresses around the dowels, although critical for the concrete around the dowels, need not be considered critical for the pavement fiber stresses, especially as the largest tension load stresses in the bottom of the pavement edge cannot be added to the largest misalignment stresses. The largest misalignment stresses at the dowels occur when adjacent dowels are misaligned in opposite direction. The largest edge stresses may occur when two or more dowels deviate in the same direction.

The modulus of support of the dowel in the concrete, or, as it will be called, "the modulus of dowel reaction K," has been given a value of 1 000 000 lb per cu in., which is thought to be reasonable for concrete pavements. Dean Grinter has suggested possible K-values in the field as low as from 3 000 to 30 000 lb per cu in. Such a suggestion is based on the assumption that the pavement deflection, as such, is reflected in the factor K. Obviously, that is not the case. The rate at which the concrete around a dowel resists the pressure of the dowel in the concrete does not change with deflection of the body of the concrete. The sub-grade influences the modulus of dowel reaction only to the extent that it permits local deformation of the concrete directly below the dowel and introduces curvature of the pavement slab. The modulus of dowel reaction, therefore, is relatively independent of the material beneath the concrete. Considering the clear relationship between modulus of elasticity of the concrete and the modulus of dowel reaction and customary pavement dimensions, the value of K for reasonable pressures would rarely be less than 25% of the modulus of elasticity.

There is nothing logical in choosing an arbitrary length of dowel embedment and presuming that such a length equals the distance to the second stress-change point, only because such a choice avoids the direct use of the factor K. The value of K is of great influence not only upon the bearing stresses around the dowel, but upon its deflection as well. The relationship between K, deflection, and bearing stress is illustrated in Fig. 16 for a 1-in. open joint, and for dowel sizes from $\frac{3}{4}$ in. to $1\frac{1}{4}$ in. round, deformed. It is evident that the deflection of the dowel for K-values less than 500 000 lb per cu in. quickly becomes so great that the dowels would be considered useless for their basic function of load transfer and would not be used. Fortunately, K would scarcely assume the low values suggested.

Inordinately high values of K would likely result in plastic deformation in the concrete due to the increased bearing stresses. High K-values (which are probably far more common in occurrence) would then be balanced auto-

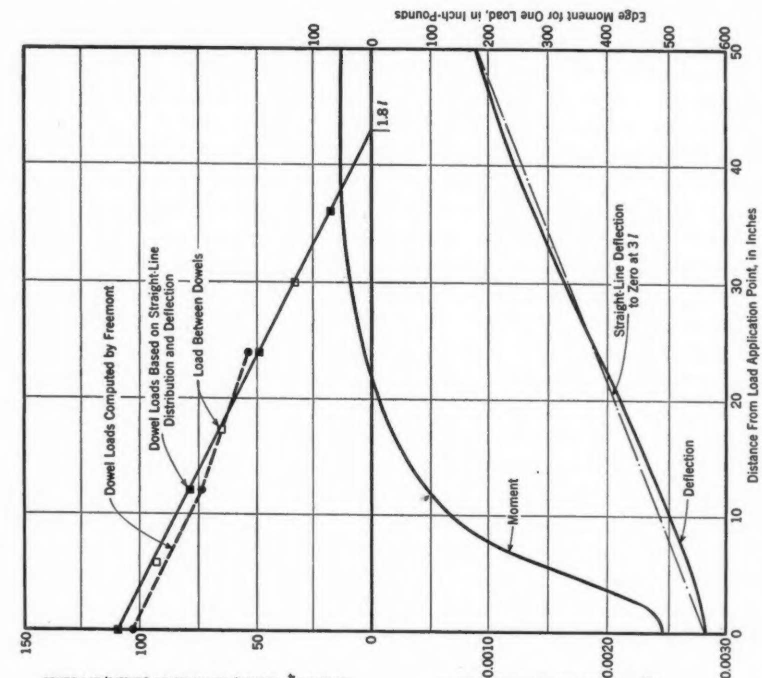


FIG. 17.—DEFLECTION, MOMENT, AND DOWEL LOADS ALONG THE EDGE FOR MODULUS OF SUB-GRADE REACTION 250 POUNDS PER CUBIC INCH; MODULUS OF DOWEL REACTION 1 000 000 POUNDS PER CUBIC INCH; AND RADIUS OF RELATIVE STIFFNESS "l" = 24 INCHES

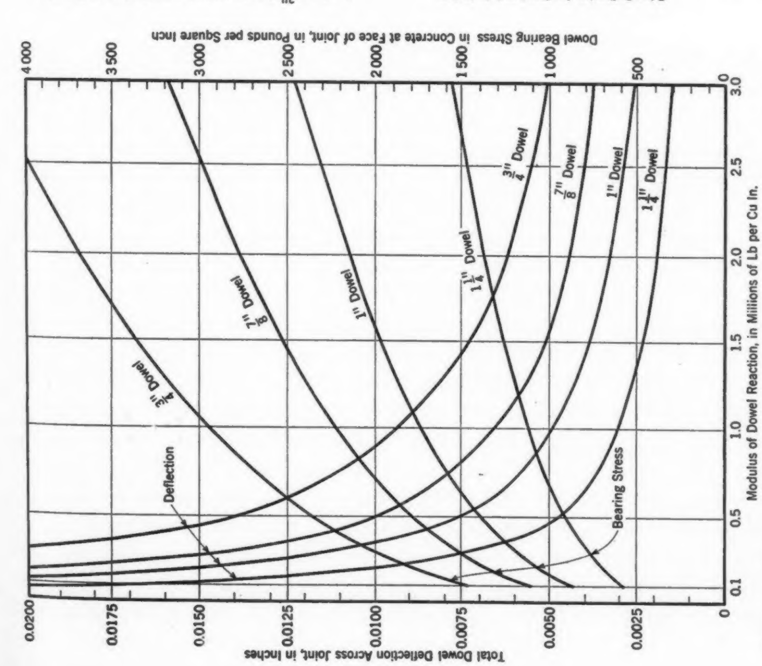


FIG. 16.—DEFLECTION AND BEARING STRESS FOR ROUND DOWELS CROSSING A ONE-INCH JOINT; 1000-POUND DOWEL SHEAR AT VARYING MODULI OF DOWEL REACTION

matically by the increased deflection and decreased load transfer accompanying a dowel of resulting longer free length. The actual bearing stress at the surface of the dowel encased in concrete may be greater than the bearing stress distributed across the diameter of the dowel by as much as 25% (see the rule for radial distribution of bearing stress in Fig. 2).

The estimate of differential warping stresses, by Dean Grinter, indicating a resultant dowel shear of the order of 2 400 lb, does not take into account the fact that the dowel shear due to differential warping would act as a lifting force on one of the slabs, resisted only by the weight of the concrete. Dowel shears considerably less than 1 000 lb for the conditions cited would then be likely to occur. Warping due to moisture differentials would probably raise the two sides of the joint alike. Curling due to temperature differential through the slab would also be likely to cause equal deflection of the two sides of the joint. Warping, therefore, scarcely needs to be considered in the design of dowels. A serious difference in level between adjacent slabs is due, more likely, to the loss of sub-grade support under one of the slabs. High dowel shears of amount equal to, or exceeding, the dowel shears computed for one dowel only according to Equation (16) will then be produced by large wheel loads; but the load-bearing capacity will be limited quickly by the unsupported pavement's resistance to bending stresses.

The effect of dowel spacing upon load transfer and stress relief was presented in the paper with three assumptions to obtain a simple, fairly representative picture, not too tedious for use in actual design. The three assumptions were:

(1) The moment diagram under a dowel load is similar to that under the wheel load;

(2) The influence of dowel shear upon pavement stress decreases with increasing distance from the wheel load to zero at a distance of 1.8 l; and

(3) A dowel directly under the wheel load remains fully effective.

Mr. Fremont has shown, quite correctly, that with a number of dowels crossing a joint the shear on the dowels nearest the wheel load is decreased. This should relieve high pressures around the dowels, but would also decrease the stress relief furnished by the dowels. Equation (16) is correct for one dowel only. However, even temporary lack of sub-grade support or voids around the dowels would immediately increase the load on the dowels nearest the wheel load. It seems wise to suggest that Equation (16) should be used for dimensioning of the individual dowel when spaced 12 in. or more apart under single tire loads. For the same reason the influence of dowels with sufficient clearance to be free to move, and some distance away from the wheel load where the pavement deflection is correspondingly smaller, would be too small to justify computation of their effect upon pavement stress.

The dowel shears computed by Mr. Fremont for a 7-in. pavement, a sub-grade modulus of 250 ft per cu in., and a 1-in. joint with five $\frac{3}{4}$-in. dowels spaced 12 in. center to center are shown in Fig. 17. Also shown are the moment diagram and the deflection diagram for a single load at the edge. The deflection diagram approximates closely a straight line with zero at a distance of

3.0 l from the load point, a condition typical for edge loadings in accordance with the diagrams developed by Dean Westergaard.[7] The moment quickly diminishes on either side of the load point and reaches a small negative maximum at a distance of about 1.8 l from the load point. Values of distributed dowel shear, based on the straight-line deflection diagram, and on the assumption of linear decrease in dowel loads to zero at a distance of 1.8 l, have been computed by the method described by Dean Westergaard,[17] adjusted to include dowel deflection. These values, plotted for the load over one, and between two, dowels, approach very near to those computed by Mr. Fremont without simplifying assumptions. The illustrated change in computed dowel shear between adjacent dowels is such that for all practical purposes dowels more than 1.8 l distant should be neglected even if the negative moment, maximum at that point, did not suggest that expedient. Therefore, computed values of dowel shear could be used for individual dowel dimensioning under conditions of reliable sub-grade support and accurate construction, especially for dowels on close spacing. The relative edge-stress relief for all dowels at variable dowel spacing, shown in Fig. 5, approximates very closely the actual conditions, expressing as it does the influence of spacing alone. The elastic characteristics of the joint and the joint opening are reflected in the basic value of shear established for a dowel directly under the load.

With the dowel shear distribution computed by Mr. Fremont, the edge-stress relief would be approximately 60% of that obtained under assumption of full effectivity of the dowel directly under the load. The actual edge-stress relief is believed to be considerably higher than that computed from distributed dowel shears. The pavement stress under the load is a function of sub-grade pressure which, in turn, is a function of pavement deflection. Dowels at considerable distance from the load point will be effective in decreasing the deflection, and therefore in affecting the distribution of sub-grade pressure near the load point, although their effect upon stress relief as gaged from the moment diagram would be to add to, rather than relieve, the pavement stress. Higher stress relief may be anticipated also, due to the true position of the dowel reaction at the edge, whereas the maximum edge stress under a wheel crossing a doweled joint occurs while the center of the contact area is some distance from the edge. The greater a concentration of effective dowel shears near the load point is assumed, the greater would be the computed shears on the dowels nearest the load point, and therefore, also, the computed stress relief. How closely the assumption of full effectivity of a dowel directly under the load will be approached with the dowel spacings that are now common remains to be established. For 18-in. dowel spacing the assumption agrees well with observed conditions. For critical comparison of various types of load transfer on different spacings, the relative, and not the absolute, value is sufficient.

The observed stress relief for the 6-in. pavement in Table 5 corresponded

[7] "Computation of Stresses in Concrete Roads," by H. M. Westergaard, *Proceedings*, Fifth Annual Meeting, Highway Research Board, 1925.

[17] "Spacing of Dowels," by H. M. Westergaard, *Proceedings*, Eighth Annual Meeting, Highway Research Board, 1928.

to a basic stress relief of 22.8% for one dowel directly under the load. To obtain this value with a computed dowel-shear distribution as presented in Fig. 17 would necessitate a dowel deflection across the joint of only 0.001 in. per 1 000 lb dowel shear, which is less than that obtainable with any possible modulus of dowel reaction for a $\frac{3}{4}$-in. dowel. The observed stress relief accordingly offers further evidence that the dowels nearest the load are more effective than the distributed dowel shears would indicate.

Thanks to the discussion, the value of the paper as an indication of conditions necessary to consider in the design of dowels across transverse joints in concrete pavements has been increased. Due to the uncertain factors upon which the mathematical analysis of a concrete pavement must still be based, it is scarcely recommended that tedious refinements be established for the detail of dowel design. The data give full recognition to the necessity of close dowel spacing for the relief of concrete pavement stresses. Future research may throw added light upon the actual deflection distribution and moments at joints so that the high effectivity of dowels near the load for pavement-stress relief may be substantiated theoretically.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TENSION TESTS OF LARGE RIVETED JOINTS

Discussion

BY E. C. HARTMANN AND MARSHALL HOLT,
ASSOC. MEMBERS, AM. SOC. C. E.

E. C. HARTMANN,[47] AND MARSHALL HOLT,[48] ASSOC. MEMBERS, AM. SOC. C. E. (by letter).[48a]—Referring to the heading, "Recommendations Applicable to Design," the writers were particularly interested in Statements (1) and (2). Regarding Statement (2) that "There is no justification for elaborate formulas to calculate the effect of rivet stagger on net section," the writers have assumed that "elaborate formulas" refer to Equation (3) taken from the specifications of the American Association of Highway Officials[12] (A. A. S. H. O.) and identical in effect with that in the A. R. E. A. Specifications. Having used this method of computing net section with satisfactory results, and having frequently tested riveted joints with efficiencies in excess of 75%, the writers decided to review the test results to check the authors' recommendations.

Figs. 25 and 26 show the types of specimens tested and Tables 32, 33, and 34 show the detailed test results. These specimens are from various investigations and do not represent a single planned series of tests. Some are too small to be considered structural joints in the ordinary sense, but these can be treated as scale models of structural joints.

In dealing with rivet sizes ranging from 3/16 in. to 7/8 in., it would be misleading to add the same fixed amount (1/8 in.) to the rivet diameters in arriving at the sizes of holes to be deducted. Therefore, all holes to be deducted were assumed to be 15% larger than the rivet diameter, a percentage which is consistent with current practice when applied to the range of sizes from 3/4 in. to 7/8 in.

Table 35 summarizes the test results and includes a parallel tabulation of the results of some of the tests made by the authors and some described in

NOTE.—This paper by Raymond E. Davis and Glenn B. Woodruff, Members, Am. Soc. C. E., and Harmer E. Davis, Assoc. M. Am. Soc. C. E., was published in May, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: September, 1939, by Messrs. Charles F. Goodrich, Frederick P. Shearwood, and Jonathan Jones; October, 1939, by Messrs. C. C. Winter, W. M. Wilson, and J. M. Garrelts; and April, 1940, by A. E. Richard De Jonge, M. Am. Soc. C. E.

[47] Research Engr., Aluminum Co. of America, Aluminum Research Laboratories, New Kensington, Pa.

[48] Research Engr., Aluminum Co. of America, Aluminum Research Laboratories, New Kensington, Pa.

[48a] Received by the Secretary March 21, 1940.

[12] "Standard Specifications for Highway Bridges," Am. Assoc. of State Highway Officials, 1935.

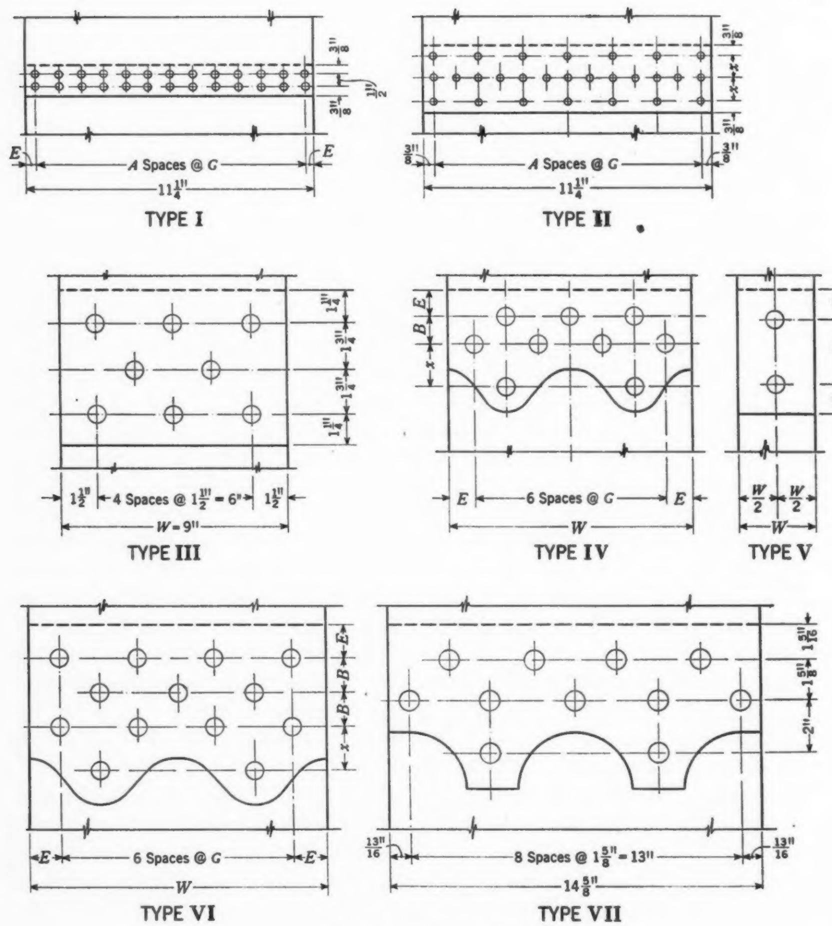


FIG. 25.—RIVETED JOINTS

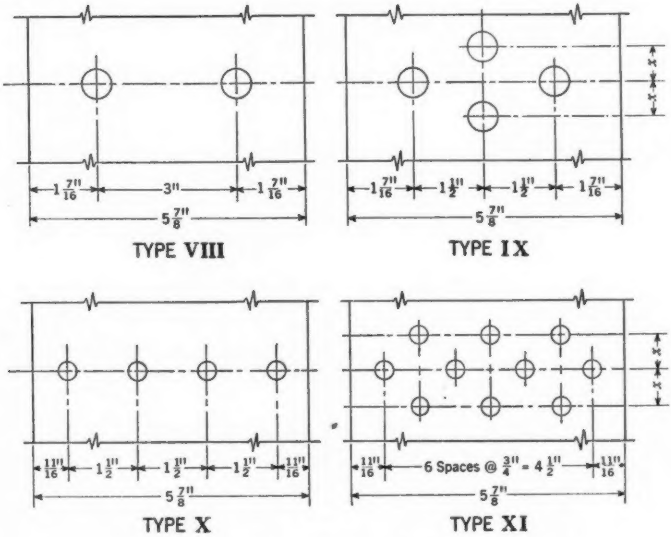


FIG. 26.—PLATES WITH OPEN HOLES OR IDLE RIVETS

TABLE 32.—CHARACTERISTICS OF TEST SPECIMENS

Type* (1)	Size of rivets, in inches (2)	MATERIAL§ Rivets (3)	MATERIAL§ Main Plates (4)	Type* (1)	Size of rivets, in inches (2)	MATERIAL§ Rivets (3)	MATERIAL§ Main Plates (4)
(a) ALUMINUM ALLOY JOINTS				(b) STEEL JOINTS			
I	3/16†	53S-W	52S-1/2H	IV(a)	5/8†	Steel‖	Steel¶
II	3/16†	53S-W	52S-1/2H	IV(b)	1/2‡	Steel‖	Steel¶
III(a)	3/4†	17S-T	17S-T	IV(c)	7/16†	Steel‖	Steel¶
III(b)	3/4†	17S-T	61S-T	IV(d)	5/16‡	Steel‖	Steel¶
V(a)	1/4†	17S-T	17S-T	IV(e)	1/2‡	Steel‖	Steel‖
V(b)	1/2†	17S-T	17S-T	IV(f)	5/16‡	Steel‖	Steel‖
V(c)	5/8†	17S-T	17S-T				
V(d)	3/4†	17S-T	17S-T				
VI(a)	3/4‡	53S-W	53S-T				
VI(b)	1/2‡	53S-W	53S-T				
VII	7/8†	53S-W	52S				

* See Fig. 25. † Single shear. ‡ Double shear. § For chemical composition and typical properties, see *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 1267. ‖ Carbon steel. ¶ Low-alloy steel containing approximately 1.45% manganese and 0.18% carbon.

TABLE 33.—RESULTS OF STATIC TENSION TESTS ON RIVETED JOINTS THAT FAILED IN THE PLATES (SEE CHARACTERISTICS IN TABLE 32)

Specimen type*	Main Plates: Tensile strength, in pounds per square inch	Main Plates: Thickness, in inches	Specimen Dimensions, in Inches (Symbols in Figs. 25 and 26): W	A	G	E	B	x	Ultimate load, in pounds	Percentage efficiency	Predicted load (A. R. E. A. method) in pounds	Ratio, predicted load to actual load
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
						(a) Aluminum Alloy Joints						
I†	35 800	0.050	11 1/4	12	7/8	3/8	1/2	....	16 050	79.5	15 100	0.94
I†	35 800	0.050	11 1/4	11	15/16	15/32	1/2	....	16 920	84.0	15 500	0.92
II†	35 800	0.049	11 1/4	14	3/4	3/8	....	1/2	16 960	85.9	16 500	0.95
II†	35 800	0.050	11 1/4	12	7/8	3/8	....	3/4	17 080	84.7	17 400	1.02
III(a)†	62 200	0.332	9	4	1 1/2	....	1 3/4	....	138 250	74.2	142 700	1.03
III(b)‡	46 800	0.388	9	4	1 1/2	....	1 3/4	....	124 450	75.9	120 900	0.97
V(a)	60 900	0.082	1	....	....	7/16	1	....	3 060	61.3	3 560	1.16
V(a)	61 400	0.064	1	....	....	7/16	1	....	2 620	66.7	2 790	1.06
V(a)	61 400	0.051	1	....	....	7/16	1	....	2 060	65.8	2 230	1.08
V(a)	60 200	0.041	1	....	....	7/16	1	....	1 670	67.6	1 760	1.05
V(b)	63 300	0.125	2	....	....	13/16	1 3/4	....	10 800	68.4	11 300	1.04
V(b)	61 800	0.103	2	....	....	13/16	1 3/4	....	7 700	60.5	9 100	1.18
V(c)	63 300	0.125	3	....	....	1	2 1/8	....	16 650	70.0	18 100	1.09
V(d)	60 400	0.185	3	....	....	1 3/16	2 1/2	....	24 300	72.3	23 900	0.98
VI(a)†	38 400	0.479	11 1/2	6	1 1/2	1 1/4	1 5/16	1 11/16	164 200	77.6	151 400	0.92
VI(b)	41 900	0.330	7 5/8	6	1	13/16	7/8	1 1/8	79 450	75.2	72 500	0.91
VII	28 600	0.500	14 5/8	8	1 5/8	....	1 5/8	2	144 000	68.8	144 900	1.01
						(b) Steel Joints						
IV(a)†	74 900	0.226	9 1/2	6	1 1/4	1	1 1/8	1 5/8	105 250	65.5	105 200	1.00
IV(b)†	74 500	0.228	7 5/8	6	1	13/16	7/8	1 3/16	97 250	75.1	91 600	0.94
IV(c)	83 800	0.143	6 3/4	6	7/8	3/4	13/16	1 1/8	67 100	82.9	55 400	0.82
IV(d)	85 400	0.149	4 3/4	6	5/8	1/2	9/16	3/4	47 900	79.3	43 100	0.90
IV(e)†	64 400	0.276	7 5/8	6	1	13/16	7/8	1 3/16	109 550	81.1	95 900	0.88
IV(f)	71 600	0.193	4 3/4	6	5/8	1/2	9/16	3/4	51 950	79.2	46 700	0.90

* See Figs. 25 and 26; one specimen tested of each type except as noted. † Two specimens tested. ‡ Three specimens tested.

TABLE 34.—RESULTS OF STATIC TENSION TESTS ON ALUMINUM ALLOY PLATES WITH OPEN HOLES OR IDLE RIVETS*

Type†	Diameter of hole, in inches‡	x, in inches†	Loads, in Pounds: Ultimate	Loads, in Pounds: Predicted**	Ratio, predicted to actual load
	(1)	(2)	(3)	(4)	(5)
VIII	25/32§	....	63 800	62 800	0.98
VIII	25/32	....	56 700	62 800	1.11
VIII	25/32‖	....	59 100	62 800	1.06
VIII	25/32¶	....	55 500	62 800	1.13
IX	25/32	1	51 300	54 800	1.07
IX	25/32	1 1/4	53 000	57 600	1.09
IX	25/32	1 1/2	51 400	61 100	1.19
IX	25/32	2 1/4	54 200	62 800	1.16
X	13/32	....	59 500	62 800	1.05

Type†	Diameter of hole, in inches‡	x, in inches†	Loads, in Pounds: Ultimate	Loads, in Pounds: Predicted**	Ratio, predicted to actual load
	(1)	(2)	(3)	(4)	(5)
XI	13/32	1/2	50 700	50 800	1.00
XI	13/32	5/8	53 600	55 000	1.03
XI	13/32	3/4	54 600	60 100	1.10
XI	13/32	1 1/8	57 200	62 800	1.10

* All plates 1/4 in.; 17S-T alloy; tensile strength, 60 500 lb per sq in. † See Fig. 26. ‡ All holes punched and open except as noted. § Drilled holes. ‖ 3/4-in. rivets, normal driving pressure. ¶ 3/4-in. rivets, excessive driving pressure. ** Predicted by A. R. E. A. method.

TABLE 35.—COMPARISON OF MAXIMUM EFFICIENCIES AND RATIOS OF PREDICTED TO ACTUAL ULTIMATE LOADS*

Number of speci-mens	Alloy	Laboratory	Maximum efficiency (percent-ages)	RATIOS, LOAD PREDICTED BY A. R. E. A. METHOD TO ACTUAL ULTIMATE LOAD		
				Minimum	Maximum	Average
	(1)	(2)	(3)	(4)	(5)	(6)
25	Aluminum	Aluminum Research Laboratories	85.9	0.91	1.18	1.02
9	Steel	Aluminum Research Laboratories	82.9	0.82	1.00	0.91
14	Aluminum†	Aluminum Research Laboratories	....	0.98	1.19	1.08
39	Steel	National Bureau of Standards‡	85.0	0.88	1.06	0.98
24	Steel	University of Illinois§	87.1	0.93	1.05	0.99
26	Steel	University of California‖	78.8	0.92	1.22	1.11

* All specimens failed by tension in the plates. † Plates with open holes or idle rivets. ‡ Based on data from "An Investigation of the Behavior, and of the Ultimate Strength, of Riveted Joints under Load," by E. L. Gayhart, *Transactions*, Soc. of Naval Architects and Marine Engrs., Vol. 34, 1926. § "Tests of Joints in Wide Plates," by Wilbur M. Wilson, M. Am. Soc. C. E., James Mather, and C. O. Harris, *Bulletin No. 239*, Eng. Experiment Station, Univ. of Illinois, Tables 6 and 8. ‖ See Tables 5 and 17 of the present paper.

the authors' references. This table shows that each group of riveted joints had one or more joints with efficiencies considerably in excess of the 75% which the authors recommend as the useful upper limit for practical design. This indicates that possibly the authors' first recommendation is too conservative. Table 35 further shows that the commonly used A. R. E. A.-A. A. S. H. O. method of calculating net section gives predicted loads which average within 11% of the actual ultimate loads for a wide variety of joints in several different materials. This spread is not excessive when compared with the accuracy expected of other design formulas such as those for column strength, allowable compression in beam flanges, and spacing of web-plate stiffeners.

The writers have yet to find any method of calculating net section of riveted joints and tension members which gives consistently better results than the relatively simple method outlined in the present A. R. E. A. Specifications. The only feature of this method which is open to serious criticism is its tendency to encourage inexperienced detailers to go too far in omitting rivets in the outside rows. It would seem that this weakness of the method could be easily overcome, without sacrificing any of the advantages of the method, by some simple restriction on the maximum spacing of rivets in the outside row. For example, a statement similar to the following, added to the article on net section in the A. R. E. A. Specifications, would serve the purpose:

> The transverse distance between rivets in the outside row shall not exceed eight times the nominal diameter of the rivet, and the edge distance for the end rivet in the outside row shall not exceed four times the nominal diameter of the rivet.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

DEVELOPMENT OF THE COLORADO RIVER IN THE UPPER BASIN

Discussion

By C. C. Elder, Assoc. M. Am. Soc. C. E.

C. C. Elder,[5] Assoc. M. Am. Soc. C. E. (by letter).[5a]—As indicated by the title, the paper includes a comprehensive picture of Upper Colorado River Basin developments and a summary (see Appendix), in readily understood form, of the status of planning and investigations for future projects. The paper's chief value may well be (as was doubtless intended by the author) to broaden the viewpoint of engineers and others interested directly and too narrowly in special projects and localities, but who so often seem "too close to the forest" to see more than a few of "the trees."

For a considerable accomplishment toward this difficult goal, the author deserves much credit; and, as regards the portion of the paper covered by the title and relating specifically or chiefly to the Upper Basin, its data and conclusions appear to conform reasonably well with official and generally accepted standards, supported by an ample direct knowledge of the subject, being therefore fairly immune to criticism. So much can scarcely be granted for the numerous references, detailed quantitative data, and generally questionable conclusions concerning the water supply of the Lower Basin. Here, material departures from reality are noted, as this has been slowly achieved in the midst of the Colorado desert by generations of difficult pioneering; after long-continued and involved litigation; through conferences and negotiations without beginning or end, and therefore timeless as all eternity (or so they seemed); with agreements and contracts for water, valued and guarded like the last drink in the desert canteen; and, above all, by about one-half billion dollars of new water-supply construction within the decade 1930–1940, practically all of which must be repaid at full interest rates and without grants, subsidies, or other financial jugglery.

The author concludes (heading "Irrigation Development") that "The

Note.—This paper by Thomas C. Adams, M. Am. Soc. C. E., was published in September, 1939, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the paper.

[5] Hydr. Engr., Met. Water Dist. of Southern California, Los Angeles, Calif.

[5a] Received by the Secretary February 14, 1940.

[Colorado River] Compact is a comparatively simple document." A mere reading of it might well seem to justify this appraisal, but not to any one who has listened to leading legal luminaries argue the pros and cons of rival interpretations of its innocent-looking phrases through six humid, but far from humorous, weeks of a Washington, D. C., summer. This argument has been interrupted occasionally during the last decade, as a result of the generated or prevailing heat, but as yet (1940) is far from being concluded.

The statement under the heading "Distribution of Water Between Upper and Lower Basins" that Table 2 "is intended to show what restrictions the Colorado River Compact places upon the distribution of the water under these conditions" (run-off as of 1922 to 1937) and the heading of the same table "* * * Distribution of Water Between Upper and Lower Basins as Required by the Colorado River Compact," are insufficiently qualified. No such restrictions or requirements as those of Table 2 are really inherent in, or definitely derivable from, the Compact; these are rather an expression very largely of "personal equation," being involved in more or less arbitrary assumptions and estimates, and in individual or regional interpretations of the Compact terms.

The Compact appears, in simple language, to allocate 8 500 000 acre-ft to the Lower Basin and 7 500 000 acre-ft to the Upper Basin. Quite improperly, in Table 2, the author assigns, for ultimate consumption, 8 800 000 acre-ft to the Upper Basin (for a year with run-off such as 1929, for example) while allotting the Lower Basin 6 200 000 acre-ft (in addition to Arizona's Gila River water use, which that State's officials insist must be called no more than 1 000-000 acre-ft annually). The Compact is certainly more subtle than has been suspected, if this result is "required" by its terms or can even be derived plausibly from them. The point is raised merely because it involves a general Compact interpretation which can be simply and briefly explained and understood, differing thus from some of the more involved hydrological problems.

The author has provided (see heading "Distribution of Water Between Upper and Lower Basins"; explanation of Column (5), Table 2) for Upper Basin development "to the limit imposed by an average use of 7 500 000 acre-ft per yr"; but it is literally true that nowhere in the Compact does the word "average" appear. It can be agreed that this usual interpretation of the Compact allocation, as adopted by the author, may finally become accepted by custom, or may prevail after long litigation and adjudication as an "equitable" interpretation. However, it is not consistent with present appropriative water-right law and practice in the Upper Basin States, where an upper diverter is not permitted to increase his claimed or decreed water right in a wet year at the expense of lower water users, merely in order to average the effects of past or future dry seasons.

If such an average allocation had been intended by the Compact allocations, presumably it would have been explicitly stated, as in Article III (*d*) with reference to the progressive cumulative 10-yr aggregate run-off as guaranteed at Lee's Ferry. It is not implied that the Upper Basin cannot, under the Compact, make use of surplus water in wet seasons, particularly to refill empty reservoirs (which holdover storage is not charged as a use until in later years it is actually diverted and consumed), as long as the Lee's Ferry flow is not

reduced below the guaranteed minimum and the Lower Basin is not prevented from receiving its full allocation. In the long run, however, such excess use as listed in Table 2 must permit less holdover storage in Boulder Canyon reservoir and therefore, in some degree, reduce the supply available for the Lower Basin. Of course, the Upper Basin has the right to assert a claim to such excess water after 1963, and then secure an allocation of it if such claim is valid and justified, but apparently it has no such right of assertion of such a definite claim at this time. However, the present specific criticism is simply that the author has adopted the Compact interpretation most favorable to his own locality, without making clear that other interpretations and quantitative results are possible and may be considered by some to be more reasonable and equitable, and to conform more strictly to the terms of the Compact.

As regards the data of Table 1, the author might be presumed to have obtained the Arizona portion from California representatives, and *vice versa* for the California "discrepancies." The new Gila Project, for example, is listed as including 2 900 000 acres but allotted only 405 000 acre-ft for ultimate annual consumption (minimum requirements there being 4.0 to 4.5 acre-ft per acre). This is clearly an individual judgment as to the probable rate of development of this desert area, but if its settlement should proceed more rapidly than thus assumed by the author, and Arizona should continue outside the bloc of Compact States, the set-up of Table 2 may prove surprisingly in error. The carefully "staked-out" future water supply now depended upon by the Upper Basin would presumably be subject to appropriation in Arizona for use on this Federal project, largely defeating the Compact's attempted long-range protection. However unlikely, this is merely another of the numerous contingencies or variations that prevent the author's computations from being, with any degree of definiteness and finality, the one set-up to meet Compact requirements. As long as Arizona is not a party to the Compact, and until the terms of this treaty are interpreted with somewhat more judicial disinterestedness than now seems the custom, a wide range of such set-ups, rather than one, appears necessary to present an adequate picture of future water supply possibilities in the Colorado River Basin.

The author views with some alarm the possibility of severe water shortages in the Colorado River Basin during future drought cycles, this fate seeming to be assigned chiefly to the Lower Basin, even as happens to lower rather than upper diverters on an irrigation ditch. Without entirely discounting this pessimism, it is believed to be subject to various qualifications. The present series of years of low run-off is a reminder of a similar but rather less severe drought period that terminated in 1905. There is some basis for the opinion that droughts as severe as the present one may have a frequency of not more than once in a century; but considering only the period of record, 1897 to 1939, the average is materially reduced below the probable long-time mean by the inclusion in the reckoning of two major droughts and only one intermediate wet cycle. The period 1905 to date (1940) probably gives a more accurate approximation to the long-time mean, and on this basis the early estimate of average virgin flow referred to in the paper (heading "Appraisal of

Past Experience") as 17 000 000 acre-ft seems not more than 2% to 3% too high.

Such average flows are of real value only when regulated by holdover storage for use during droughts. The effective active storage of Lake Mead will be increased greatly, long before needed by the Lower Basin for water supply, by the construction of additional reservoirs in the Upper Basin. The Bridge Canyon Dam will promptly cut off practically all silt inflow to Lake Mead for many years into the future, and every up-stream reservoir will have the same effect in some degree. Upper Basin and canyon reservoirs will aid in flood-control regulation, permitting the ultimate reduction of the present Lake Mead flood-control reserve of 9 500 000 acre-ft to a nominal amount. Below Lake Mead, Parker and Bullshead reservoirs will conserve winter power water. Instead of the author's 20 000 000 acre-ft of available holdover storage, there will finally be functioning (by the time needed) between 30 000 000 and 40 000-000 acre-ft of active regulatory capacity. The outlook thus appears much less gloomy—in fact, very satisfactory.

Similarly, on the credit side of the ledger, Item 13, Table 1, for silt sluicing at Imperial Dam has shrunk to a negligible amount with the rapid clearing of the Colorado River water that has already occurred there. Doubt has also been expressed that sufficient discount has been allowed for the high elevations at which proposed large transmountain diversions must be made, reducing materially the portion of the available run-off that can be so diverted. Other such proposed transmountain projects involving high pump lifts must, even in drought years, allot sufficient water (and, incidentally, additional water above this minimum) to the canyon power plants to generate the power required for pumping if commercial power and resulting revenues to help pay for such projects are to be produced. Such natural and economic restrictions on excess Upper Basin diversions may well appear more reassuring to Lower Basin water users than even the guarantees of the Colorado River Compact.

The author's theory (heading "Distribution of Water Between Upper and Lower Basins"; explanation of Column (11), Table 2[6]) of large errors in Boulder Canyon reservoir areas and capacities, stated to be of the order of 5% to 10%, or several thousand acres, seems absolutely untenable, even in that difficult canyon region, in view of the high engineering standards of the Bureau of Reclamation, with Boulder Canyon reservoir itself the best possible proof that these standards have been maintained. A series of air-survey maps, flown at varied water-surface elevations, should be able to answer this question of stage-volume relationship quickly and with conclusive accuracy. Meanwhile, it is considered that the large, unaccounted inflow to Boulder Canyon reservoir cited under "Distribution of Water Between Upper and Lower Basins" (discussion of Column (10), Table 2) can be explained more satisfactorily as bank storage, since the allowance of 10-ft depth over increments of wetted area can be called "generous" only in comparison with earlier estimates of 3 ft for bank storage. At Jackson Lake reservoir, with a capacity of 847 000 acre-ft, but with a water-level rise of only 8% of that at Boulder Canyon reservoir, a bank storage inflow from 10% to 15% of the reservoir capacity has been measured. In any case,

[6] Also personal letter from author dated February 3, 1939.

whether Lake Mead is larger than assumed, or has unexpectedly large bank storage, the water supply on the Lower Colorado River will be aided materially, in time of drought, by such an increase of active or effective storage.

The writer feels much sympathy for the paper's concluding plea (see heading "Conclusions") "for the use of available water supplies to expand [Upper Basin] communities already established" and "in ways and places that will, most effectively, build communities with a broad industrial base" in addition to agriculture. This need is called a "further justification * * * for the relatively high cost of these [proposed transmountain diversion] projects and the initial financial subsidization they need," and with the latter statement little disagreement will be found, even in the Lower Colorado Basin and among engineers (those least radical or politically minded of mortals). Search the paper as one will, however, no reference to, or explanation of, any primary or initial justification of such "subsidization" can be found to which this appealing "further justification" must presumably be intended to be added in order to give to such proposed projects at least semblance or approximation to economic feasibility. The promotion of such large-scale "subsidization" of water exports is the only factor ever seriously viewed by Lower Colorado River Basin water users as constituting a genuine threat to the permanent safety of their future water supply. To engineers in general, it remains an unpleasant and distasteful idea, foreign to their mental processes and professional standards.

Corrections for *Transactions:* In *Proceedings*, September, 1939: page 1220, line 27, change "4 200 000" to "4 400 000"; and page 1223, line 12, delete the sentence beginning "Lake Mead in * * *."

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FUNCTIONAL DESIGN OF FLOOD CONTROL RESERVOIRS

Discussion

By Messrs. J. O. Jones, and W. C. Hammatt

J. O. Jones,[25] Assoc. M. Am. Soc. C. E. (by letter).[25a]—This paper is a distinct contribution to the literature on flood routing. It recognizes the fact that the storage capacity required is dependent, chiefly, upon the total volume of the flood, and only in a minor degree upon the flood distribution.

When the authors' method is applied to the problem of estimating the storage capacity required for a given maximum flood, with a given maximum outflow, a fairly large range of values of volume is obtained, depending upon the assumed duration of the equivalent uniform flood. This may be seen by applying Equation (2) and Table 2 to a concrete example.

The two floods shown in Fig. 10 have the same volume (140,000 acre-ft) but are markedly different in distribution. The exponent, m, in Equation (1) is 2.18. Hence, the values shown in Table 10 are obtained. The variation in

TABLE 10.—Design Flood, Marais des Cygnes

Item	Description	Duration T, in Days			
		4.0	4.8	5.6	6.4
1	Equivalent uniform flood, in thousands of acre-feet	35.0	29.2	25.0	21.9
2	Outflow ratio, x	0.5	0.6	0.7	0.8
3	Detention ratio, d	0.585	0.498	0.409	0.315
4	Storage (S) required, in thousands of acre-feet	81.9	69.7	57.2	44.1
5	Maximum depth, h, in feet	43.5	40.4	36.8	32.8

the results may be inconsequential, in view of the more or less speculative nature of the design flood. The percentage variation in the required storage, however, is considerable. The step method as applied herein by the writer indicates

Note.—This paper by C. J. Posey, Jun. Am. Soc. C. E., and Fu-Te I, Esq., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1939, by Edward Soucek, Jun. Am. Soc. C. E.; March, 1940, by Messrs. J. C. Stevens, Edward J. Bednarski, Ronald A. Kampmeier, and Edgar E. Foster; and April, 1940, by Messrs. Sherman M. Woodward, M. Kindinger, P. Wilhelm Werner, and Waldo E. Smith.

[25] Prof. of Hydraulics, Univ. of Kansas, Lawrence, Kans.

[25a] Received by the Secretary March 18, 1940.

that an equivalent uniform flood of 28,000 acre-ft per day with a duration of $T = 5$ days will agree exactly with design flood No. 1 but will underestimate, slightly, the required capacity for flood No. 2. Hence, it appears that a criterion is needed, which will indicate, at least approximately, the value of T in Equation (2).

It may be that a criterion can be obtained graphically. As a matter of fact, the volume of storage required is represented by the area between the flood

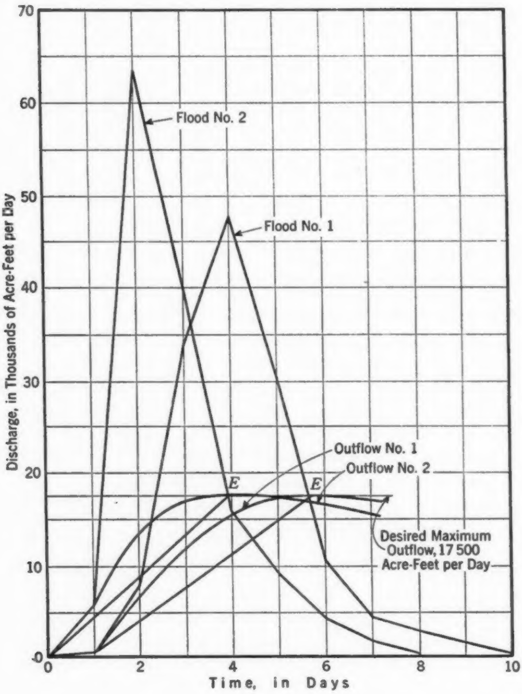


FIG. 10

hydrograph and the outflow hydrograph. Although the outflow curve cannot be obtained, except by the step method or its equivalent, a fair approximation may be had by considering the outflow curve to be a straight line drawn from zero to the point of maximum outflow (line $E\,E$, Fig. 10). The area between the straight line and the flood hydrograph overestimates the required capacity in every case. In the present example (Fig. 10) the excess is 19% for flood No. 1 and 14% for flood No. 2. An improvement may be had by noting that the outflow hydrograph is quite flat at the maximum point. By sketching a curve, which is required to pass through the points E, and which retains the characteristic shape of the outflow hydrograph, one may obtain a result quite close to that obtained by approved step methods.

W. C. HAMMATT,[26] M. AM. SOC. C. E. (by letter).[26a]—There seems to be a tendency in recent years to go into intricate and exact mathematical analyses of problems which are fundamentally of a practical nature. This tendency causes one to lose sight of, or at least to have one's vision obscured toward, the fundamental principles involved.

In the matter of the design of storage for protection against flood damage, certain factors are taken into account, any one of which may be the ruling one, depending on the facts in the particular case. These factors are as follows: (1) The expected flood flow of the stream, in intensity, duration, and frequency; (2) possible storage capacity of the proposed reservoir; (3) the capacity of the stream bed below the reservoir outlet to handle flows without flood damage; and (4) the economics of the situation—that is, the comparative study of possible property damage from flood peaks, and the cost of protective storage. In spite of the methods developed in recent years, factor (1) is still an uncertain one, based on historical analysis of the watershed itself and of others with similar characteristics; and its uncertainty has been demonstrated frequently by the occurrence of floods far in excess of the predictions developed by customary methods of analysis. Factors (2) and (3) are capable of practically exact computation. Factor (4) is capable of exact analysis under existing conditions, but is subject to the uncertainty of development and values at a future date.

Beginning with factor (4), the economic study, and calculating the maximum cost that can be assumed for flood protection, considering that unlimited storage capacity can be developed, the computer works back through factors (3), a constant, and (1), an estimate, to obtain factor (2), the desired solution. It is a fundamental principle that the solution of a problem, wherein one or more of the factors are inexact, cannot itself be exact. It follows that, since factor (1) is an estimate only, the engineer can only design to meet this assumed condition, and that simplified methods of computation may be used with as much likelihood of success as more exact mathematical ones.

If the possible storage capacity is limited to less than that necessary for complete control, the problem resolves itself into a mere determination of the rectangular outflow to utilize the entire storage capacity available, always assuming that the benefits derived by this regulation are in excess of its cost. Since the total quantity, rate, and duration of the inflow are indeterminate, a graphical solution, preferably by an assumed mass curve based on historical data, is sufficiently accurate. There is nothing new in this method of treatment, the only recent development being the improved methods of flood prediction, and the writer believes that further refinement is not consistent with the uncertainty of the basic data.

[26] Civ. and Cons. Engr., Los Angeles, Calif.

[26a] Received by the Secretary March 25, 1940.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

SEWAGE DISPOSAL PROJECT OF BUFFALO, NEW YORK

Discussion

BY THOMAS M. NILES, M. AM. SOC. C. E.

THOMAS M. NILES,[5] M. AM. SOC. C. E. (by letter).[5a]—The tunnel crossing under the Buffalo River and the main crossing under Black Rock Harbor to the sewage treatment plant are described briefly in this paper. Two additional crossings, not mentioned by name in Mr. Greeley's paper, were built under Cazenovia Creek and Scajaquada Creek. Structures of this kind are frequently required in connection with sewer work.

Black Rock Harbor is a relatively short navigable canal along the east bank of the Niagara River which carries traffic past the rapids at the head of the river. At the point of crossing it has a total width of about 400 ft, a channel width of about 250 ft, and a channel depth of about 20 ft. To allow for a future navigable depth of 27 ft, the War Department requested a clear depth of not less than 32 ft over the crossing structure, measured from low water datum. This was the only one of the four stream crossings at which active navigation had to be maintained during construction.

Although the Buffalo River is considered navigable as far as Bailey Avenue, the point of the sewer crossing, it is not now (1940) used this far up. The stream at this point is some 200 ft wide and 5 to 7 ft deep below low water. To provide for future navigation, the required depth over the crossing structure was 30 ft. Actually, this crossing was constructed in rock tunnel at a greater depth.

Cazenovia Creek and Scajaquada Creek are about 120 ft wide and 50 ft wide, respectively, and are both about 10 ft deep. The clear depth necessary was 14 ft for the former and 16 ft for the latter. Although both may be considered navigable, no development for deep draft vessels is contemplated.

The crossings were designed to provide velocities sufficient to avoid deposition of organic solids at minimum flows and sufficient to scour sand and grit at maximum flows, with reasonably low loss of head. Other controlling factors

NOTE.—This paper by Samuel A. Greeley, M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: November, 1939, by C. A. Holmquist, Esq.; and April, 1940, by Messrs. Arthur J. Bulger, and C. R. Velzy.

[5] Res. Engr. Representative, Greeley & Hansen, Buffalo, N. Y.

[5a] Received by the Secretary March 11, 1940.

were simplicity of construction, simplicity of operation, ease of inspection and maintenance, and the provision of facilities for cleaning if found necessary.

Two-pipe conduits were used for all of the crossings. Those for Cazenovia Creek and Scajaquada Creek are circular reinforced concrete pipes encased in concrete. In the Buffalo River Crossing, the smaller pipe is circular and the larger a modified rectangular section with the longer dimension vertical with a semicircular crown. The harbor crossing consisted of two pipes of equal diameter. For all except the harbor crossing, where flow is controlled manually by sluice gates, dry weather flow is confined to the smaller pipes by means of weirs.

TABLE 11.—Design Data for Stream Crossings, Buffalo Sewage Disposal Project

Crossing	1940 Sewage Quantities, in Million Gallons Daily			Pipe Diameters, in Inches		1940 Velocities, in Feet per Second					Estimated loss of head, in feet, at maximum flow
						Smaller Pipe				Maximum velocities, larger pipe	
	Minimum	Average	Maximum	Smaller	Larger	Minimum	Average	Maximum	At maximum flow		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
Cazenovia Creek......	5.5	11	62	27	54	2.1	4.3	7.5†	3.9	5.0	1.0
Scajaquada Creek.....	9	19	97	36	60	2.0	4.2	7.7†	5.9	5.6	1.0
Buffalo River........	7	14	87	36	*	1.5	3.0	5.5†	4.2	4.3	2.3
Black Rock Harbor...	66	132	566	96	96	1.0	2.0	8.7	8.7	8.7	5.5

* Special shape (48 in. by 77 in.), area = 24 sq ft.

† Maximum velocities occur in the smaller pipes just before the larger pipes come into service. Advantage is taken of an available head somewhat higher than the head available at maximum total flow.

Table 11 presents data on sewage quantities, pipe sizes, velocities, and estimated loss of head for the four crossings. In the two creek crossings the pipes are of the same cross sections throughout their lengths and the inlet and outlet portions are sloped down from inlet chambers and up to outlet chambers which meet the inverts of the connecting sewers. The river and harbor crossings have vertical inlet and outlet shafts, and in the case of the river crossing the uptake pipes in the outlet shaft are reduced in size to 30-in. diameter and 48-in. by 54-in. rectangular to provide slightly higher velocities than in the horizontal part of the crossing.

Inlet and outlet chambers for the creek and river crossings are provided with removable slabs to permit introduction of dewatering pumps and other equipment which may be useful in connection with inspection and cleaning. During dry-weather flow, the larger pipes of the creek crossings may be pumped out and inspected without disturbing the flow in the smaller pipes. Inspection of the smaller pipes involves construction of temporary dams to divert the flow through the larger pipes. Sluice gates are provided at both the inlet and outlet ends of the river and harbor crossings to permit diversion of flow to either pipe while the other is being inspected.

The outlet structure of the Buffalo River crossing includes a separate access shaft between the two riser pipes sufficiently large to accommodate a dewatering

pump and other equipment that may be needed. Manholes at the bottom of this shaft permit access into the horizontal crossing pipes, and gate valves are provided for draining the pipes into the shaft.

The harbor crossing riser pipes are within the inlet chamber of the main pumping station at the treatment plant. They are vertical tubes, 8 ft in diameter, opening into surge chambers with an overflow connection to the river. These tubes extend 2 ft below the inverts of the horizontal crossing pipes to form pockets for the collection of large stones which may not be carried upward, and from which material may be periodically cleaned by means of a clamshell bucket. A portable dewatering pump is provided to facilitate inspection and cleaning.

Construction of the two creek crossings was by open cut with cofferdams in two sections, confining the stream flow to the inactive half of the work. Aside from occasional flooding from high water, no unusual difficulties were experienced.

Alternate methods of construction, open cut or rock tunnel, were available to bidders on the Buffalo River crossing. The low bid was for rock tunnel and the contractor exercised his option to construct the tunnel at a lower elevation than shown on the plans in order to obtain more favorable tunneling conditions. Even so, it was found necessary to install timbering for roof support. Occasional grouting of seams successfully controlled the flow of water into the work.

Borings had shown the material at the site of the Black Rock Harbor crossing to be unsuitable for tunneling. Accordingly, the construction method available was open cut, either by cofferdams or by subaqueous methods. Although proposals were received on both methods, the low bidder's choice of the subaqueous method involved less difficulty in maintaining navigation during the progress of the work. Considerable additional dredging would have been required to provide sufficient widths and depths of channels at either side had cofferdams been used throughout.

Approximate average construction costs for the four stream crossings, including inlet and outlet structures, were as follows:

Crossing	Total	Per foot of length
Scajaquada Creek	$ 15,000	$150
Cazenovia Creek	27,000	150
Buffalo River	75,000	300
Black Rock Harbor	425,000	800

The crossings were placed in operation in June, 1938, and have given satisfactory service. No operating difficulties have been encountered and there has been no need for cleaning.

The foregoing discussion of one feature of the work on the Buffalo project is perhaps in more detail than the relative importance of this feature should warrant. Nevertheless, it has seemed that the material presented may be of value in acquainting the reader with some of the detailed considerations which entered into the design. The paper itself is an admirably complete and concise review of the essential features of the project and contains sufficient detailed information to present a clear understanding of the problem at Buffalo and the manner in which it was solved.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

RELATION OF THE STATISTICAL THEORY OF TURBULENCE TO HYDRAULICS

Discussion

BY MESSRS. PAUL NEMENYI, AND BENNIE N. NETZER

PAUL NEMENYI,[45] ESQ. (by letter).[45a]—The pioneer work done by Professor Kalinske in the experimental investigation of open channel turbulence and other turbulence problems pertinent to hydraulic engineering will stimulate many research workers to further inquiries. Eventually, it will also help practical hydraulic engineers to obtain a deeper insight into the fluid motion phenomena with which they deal in their daily work.

Of the many problems with which the paper deals, the writer proposes to discuss mainly one: The question of how a high rate of energy dissipation may be attained under various given conditions. The author appropriately distinguishes two main practical problems: (*a*) The disposition of energy dissipators all along a conduit or a channel in order to convert potential energy into turbulent energy, rather than into kinetic energy of the main longitudinal flow—or, in other words, to prevent the developing of large velocities; and (*b*) energy dissipation at the end of a chute to convert kinetic energy of the main flow into the more dispersed form of turbulent energy and into heat—that is, to destroy the large velocities present.

For several years, the writer has been interested in problem (*a*), which is important in the design of fishpasses, timber-floating channels, and sometimes pipes of surge tanks, and is related also to spillway design. He cannot agree with the author's opinion that quantitative turbulence research in its present stage can give suggestions or information relevant to the solution of this problem. The author bases his opinion primarily upon a formula for the rate per unit length at which potential energy is converted into turbulent energy in a circular pipe. The formula may be written in the following form:

$$\text{Rate of energy conversion varies as } \int_{y=0}^{r} \rho \sqrt{\overline{v^2}}\, l \left(\frac{dU}{dy}\right)^2 y\, dy$$

NOTE.—This paper by A. A. Kalinske, Esq., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Hunter Rouse, Assoc. M. Am. Soc. C. E.; February, 1940, by Messrs. Martin A. Mason, and J. C. Stevens; March, 1940, by Boris A. Bakhmeteff, M. Am. Soc. C. E.; and April, 1940, by Messrs. Clyde W. Hubbard, John S. McNown, and Samuel Shulits.

[45] Research Fellow, Iowa Inst. of Hydraulic Research, Univ. of Iowa, Iowa City, Iowa.

[45a] Received by the Secretary March 4, 1940.

He concludes that in order to secure highly effective transformation of potential energy into turbulent energy, l, the size of eddies (being proportional to l) should be made as large as possible. Even apart from the fact that, as this discussion will show, the reliability of the foregoing energy conversion formula has very substantial limitations, the conclusion of the author is not convincing because in a turbulent flow v and $\frac{dU}{dy}$ are modified as soon as l is modified. All characteristic quantities in the integral being inseparably related, it cannot be decided without further study whether arrangements which produce large values of l would really tend to increase the rate of energy conversion. Indeed, there are good reasons for the assumption that the highest effectivity of energy dissipation can be obtained entirely without resorting to large turbulence eddies.

With little exception statistical inquiries into turbulent flow have been concerned with cases in which the average flow (for a sufficiently long period) is steady, straight, and uniform. The foregoing formula seems to have these limitations. Obviously no rough conduit or channel can strictly satisfy these conditions; and as the size and efficiency of the "roughness" increases, local deviations of average flow from the straight line will also increase in importance. This is one reason for the inadequacy of present-day turbulence statistics in producing a satisfactory treatment of energy dissipators. But even if turbulence research will extend to far more varied phenomena, however, energy dissipation studies will always need to go beyond turbulence study and go back to the more general concept of vorticity, supplemented by such concepts as boundary layers and discontinuity surfaces. The following remarks on the different possibilities of energy dissipation in long conduits will trace (aided by these concepts) the transformation of potential energy into the kinetic energy of the turbulence eddies, thus indicating the conditions under which this kind of energy can be generated at a high rate.

The first result of such a study is that, the shape of the "roughness" obstacles being given or chosen, the spacing must be recognized as a factor of equal consequence along with the size (height), which many times even today is believed to be the all-important factor. Figs. 10 and 11 illustrate this fact in its simplest and most marked instance. Fig. 10(a) shows that obstacles, placed

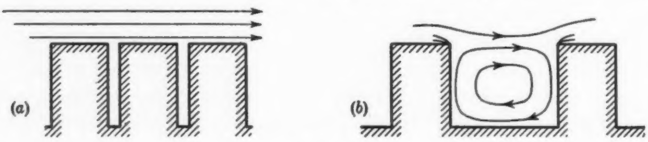


FIG. 10

so closely together that only very narrow slits remain between them, fail to disturb the boundary layer to any noticeable extent. In such slits the fluid is very nearly at rest, the pressure being the same as in the undisturbed boundary layer. The "obstacles" have scarcely any measurable energy dissipating influence upon the main flow. (Obviously, the known use of piezometric holes is based on this fact and in certain experiments piezometric holes have been actually replaced by slits in the cross-section direction.) In Fig. 10(b) the space

between consecutive obstacles is approximately the same as the height of obstacles. The slowly moving material in the boundary layer cannot bridge the gap between the two obstacles; instead the faster moving material from the inner stream transfers some of its momentum to the neighboring fluid in the gap, or "bay." This in turn, by continuity, causes a backward stream near the bottom of the "bay." Thus a vortex, as indicated in Fig. 10(*b*), is generated. However, if, as assumed, the "bay" has a square or nearly square cross section, the vortex is locally fixed (trapped) and stable. As a result, it does not emit vorticity of any considerable amount into the inner stream and remains a poor

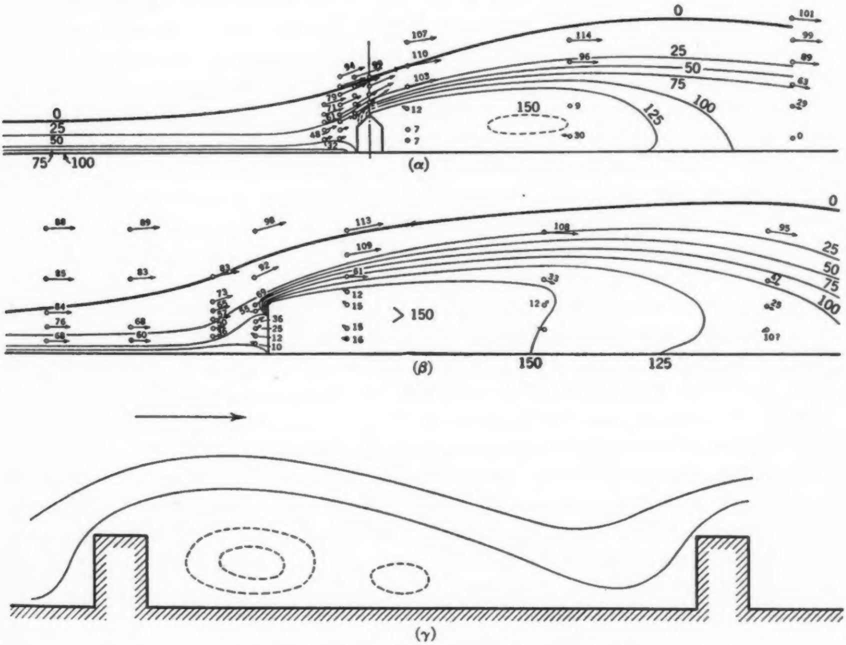


FIG. 11.—CURVES OF EQUAL TOTAL HEAD LOSS

energy dissipator although the height of the obstacle relative to the total width of the conduit may be very considerable. Only if the Reynolds number computed from the size of obstacle and the velocity of the main flow is extremely high may such a vortex of square cross section become unstable and thus become a more efficient energy dissipator; but no direct experimental evidence is available to support this view.

For longer "bays" the situation is substantially different. Obviously, a very long oval-shaped vortex is unstable; it will give place to a more complex and less stable pattern of flow. Experiments of H. Schlichting[46] indicate, however, that the maximum energy dissipation efficiency of the type of roughness in question is reached only when the spacing of obstacles is approximately

[46] "Experimentelle Untersuchungen zu dem Rauhigkeits-problem," by H. Schlichting, *Ingenieurarchiv*, Vol. 7, 1936, p. 1.

ten times their height. To understand this condition better one should first examine the case of the stream field in which there is only a single obstacle upon a smooth boundary. The writer has studied the stream fields experimentally for various shapes of individual obstacles.[47] Figs. 11 (α) and 11 (β) show the results for obstacles extending across the entire breadth of the channel, their cross sections being comparable to those under discussion. (In these curves, the arrows and corresponding numerals indicate velocities expressed as percentages of the undisturbed velocity.) It will be seen that the obstacle deflects the stream considerably, giving to it a momentum toward the inner part of the channel. Thus the region that is the seat of considerable energy dissipation extends far into the inner part of the flow. The lee of the obstacle, on closer investigation, showed velocity fluctuations approaching the same order of magnitude as the local average velocities given in Figs 11 (α) and 11(β). The unsteady part of the vorticity corresponding to these fluctuations is carried far into the inner channel and is the main source of energy dissipation there. If one now wishes to transform (at the highest possible rate) the potential energy all along the channel into such vorticity by the use of perpendicular obstacles, one must obviously place them equidistant, not too far from each other, but far enough to make the development of a considerable forward velocity on the windward side of each obstacle possible (Fig. 11(γ)). Without such a forward velocity the momentum necessary for the development of the large leeward vorticity would not be available. This reasoning explains the experimental finding by Schlichting concerning the optimum spacing. As his results indicate, the order of magnitude of optimum spacing remains roughly the same if, instead of obstacles extending across the breadth of the channel, separate roughness elements such as hemispherical shapes are used. Of course, if the individual obstacles are not all of the same size and shape the question of optimum spacing for attainment of energy dissipation is more complicated. However, it is obvious that as long as the mechanism of flow, vortex formation, and energy dissipation remain the same as described herein, no substantial improvement beyond that attained for the foregoing particular obstacle with its optimum spacing is attainable.

The Belgian hydraulic engineer, G. Denil, should be credited with showing, through his fishpass designs of 1909, that certain oblique obstacles, closely spaced, can dissipate energy in a channel at a higher rate than perpendicular or haphazard roughness elements, and with having gradually improved his system to a high degree of efficiency.[48] The principle implicit more or less in all Denil channels, and embodied with particular perfection in his more recent designs, is the following: Oblique obstacles must be shaped and arranged in such a manner that, instead of deflecting the stream inward (away from the channel sides) as the aforementioned obstacles of ordinary efficiency do, they cause the fluid to enter the "secondary channels" between them. These secondary channels, each formed by two consecutive obstacles and by a part of the

[47] "A New Device for Direct Stream Field Studies and Its Applications," by P. Nemenyi, *Ingeniørviden-skabelige Skrifter No. 39*, Copenhagen, 1935.

[48] "La Mecanique du poisson de rivière; qualités nautiques du poisson; ses methodes locomotrices; ses capacités; ses limites; resistances du fluide; effet de la vitesse; de la pente; resistance de seuil," by G. Denil, Bruxelles, 1938; see also "Fish Movements and Fishpasses," by P. Nemenyi (a review of G. Denil's book), *Nature*, September, 1939.

main channel surface, must be shaped in such a manner that they offer little resistance to the flow of fluid through them as well as against its entrance into them. If these conditions are fulfilled, obviously strong secondary currents, only moderately turbulent, will reissue into the main flow. This re-entrance should occur abruptly—that is, at an angle of nearly 90°. As will be seen, the transformation into turbulent dispersed energy for conduits of this kind is concentrated for the greatest part in the vicinity of the areas where the secondary currents re-entering strike the main flow. In order that the gaps between these areas may be as small as possible, the obstacles should be as thin as practicable.

Since practically all of the designs of G. Denil made for various special purposes are too elaborate to be entirely economical, the writer has endeavored (first in a joint research with C. M. White,[49] and later during his work at the Iowa Institute of Hydraulic Research) to incorporate these principles of energy dissipation into new designs sacrificing as little as possible in efficiency while obtaining considerable simplification. Fig. 12[49a] illustrates three of the designs

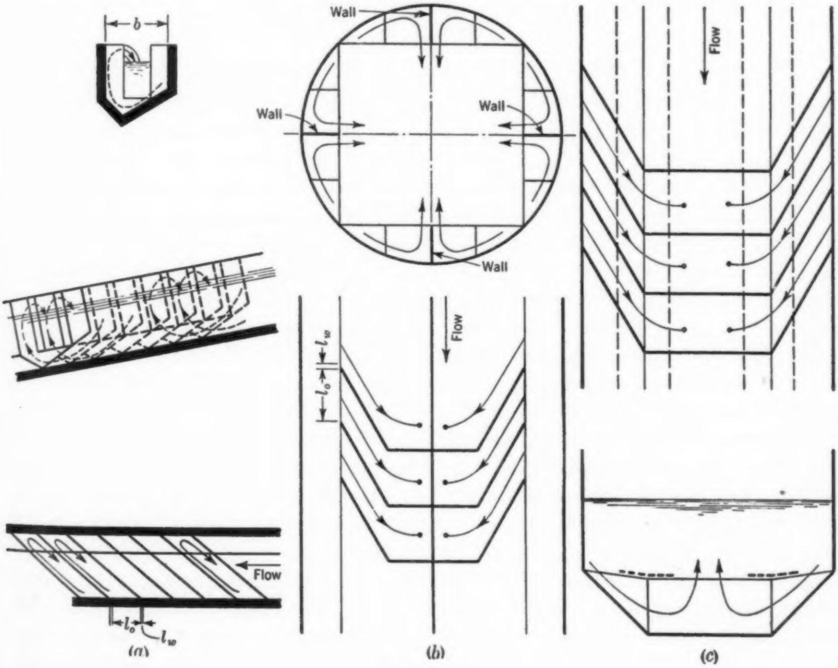


Fig. 12

developed—one of them a result of the London studies, the others of the Iowa studies. (It will be observed that the two latter designs (Figs. 12(*b*) and 12(*c*))

[49] "Report on Fishpass Research," by P. Nemenyi and C. M. White, mimeographed (printed report in *Journal* of the Institution of Civil Engineers is being prepared); see also: "Steep Channels Fitted with Energy Dissipators," by P. Nemenyi, *Transactions*, A. S. M. E., Vol. 60, 1938, Page A-118 (*Journal of Applied Mechanics*).

[49a] Correction for *Transactions:* In Figs. 12 and 13 change "*l*" to "*b*."

have almost exactly the same kind of baffles, but in one case are adapted to the conditions of a circular pipe and in the other to an open channel with a trough-like cross section.) These simple designs with the directions of flow traced in them well illustrate the foregoing principles of energy dissipation.

While testing these and similar fishpasses the writer found the following quantitative explanation for the high effectiveness of this type of energy dissipating device. Suppose the free width of one secondary channel, measured in the direction of the axis of the main channel, is b_0; the wall thickness of secondary channels (measured in the same manner) is b_w; then $b_0 + b_w$ is the spacing of obstacles along the main channel. Applying the momentum theorem to a fluid prism (Fig. 13)[49a] of length $b_0 + b_w$ and of cross section f_0 corresponding to

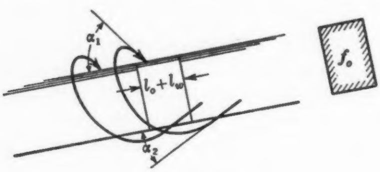


FIG. 13

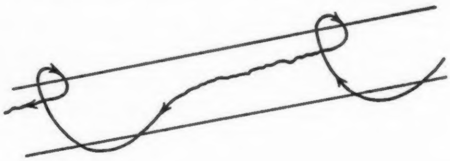


FIG. 14

the inner channel (free cross section), let S be the slope of the main channel, or the pressure loss per unit length if a closed conduit is being studied. (More exactly: $S = \sin \alpha$, in which α is the angle of the channel axis with the horizontal. The use of the approximation $S = \tan \alpha$ is acceptable only for slopes of 10° or flatter.) The force acting on the prism is then equal to $(b_0 + b_w)\,\gamma\, S$. This must be equal to: The momentum transferred to the prism by a re-entering secondary current, $F_{n,r}$; minus the momentum transferred to the secondary current issuing from the prism, $F_{n,i}$; plus the forces of turbulent fluid friction. (Obviously one and the same prism of the length $b_0 - b_w$ in the main channel receives fluid from one, and sends fluid into another, secondary channel; this makes no difference, however, provided that the regime of flow on the average is uniform, as assumed herein.) These latter forces result from the relative longitudinal velocity components between the flow in the main channel and the secondary current at its issue $(F_{t,i})$, along its course $(F_{t,c})$, and at its reissue $(F_{t,r})$. (In well-designed conduits $F_{t,r}$ is usually either 0, as in Figs. 12(*b*) and 12(*c*), or negative, as in Fig. 12(*a*). In the formula—$(b_0 + b_w)\, f_0\, \gamma\, S = F_{n,r} - F_{n,i} + F_{t,i} + F_{t,c} + F_{t,r} = F_n + F_t$—the subscript n indicates the longitudinal forces corresponding to the normal, perpendicular momentum transfer, whereas the subscript t refers to turbulent friction forces. Although the latter derived their existence from highly complicated phenomena of turbulence, which for the time being are not accessible to any mathematical treatment, the former can be computed by applying the general expression of concontinuous momentum transfer[50] to the present problem as follows:

$$F_{n,r} = \frac{\gamma}{g} Q_s \sin \alpha_1\, U = \frac{\gamma}{g} \sin \alpha_1\, f_s\, U\, V_s \ldots\ldots\ldots\ldots (31a)$$

[50] For a graphic explanation of this simple, fundamental relation, see "Mechanics of Turbulent Flow," by Boris A. Bakhmeteff, Princeton University Press, 1936, pp. 38–39.

and

$$F_{n,i} = \frac{\gamma}{g} Q_s \sin \alpha_2 \, U = \frac{\gamma}{g} \sin \alpha_2 f_s \, U \, V_s \ldots\ldots\ldots\ldots (31b)$$

in which g = gravitational acceleration; Q_s, V_s, f_s = rate of flow, average velocity, and normal cross section of the secondary channel; U = average longitudinal velocity of the flow in the main (free) channel section; α_2 = angle of entrance from the main channel into the secondary current; and α_1 = angle of reissue of secondary current into the main channel.

Per unit length of the conduit, therefore:

$$\gamma f_0 S = \rho \frac{f_s}{b_0 + b_w} U \, V_s (\sin \alpha_1 - \sin \alpha_2) + \frac{F_t}{b_0 + b_w} \ldots\ldots (32)$$

$\rho = \frac{\gamma}{g}$ being the specific mass of the fluid. If there is more than one secondary channel in the same section of the conduit the first term must be multiplied by their number, n. (In Fig. 12(a), n = 8; in Fig. 12(b), n = 2.)

Theoretical considerations, as well as experimental facts, indicate that the first term is much larger than the second, for a well-designed channel. The structure of this first term contains much of what is essential for the understanding of this system of energy dissipation, particularly if it is compared with the corresponding, well-known formula for channels or conduits of ordinary type.

For a prismatic fluid body in the free cross section of the channel, with cross section f_0, and of unit length, it is generally accepted that in smooth channels as well as in rough channels of the usual types the following relation (a generalized form of Equation (2)) is valid:

$$\gamma f_0 S = \rho P_w \overline{u} \, \overline{v} \ldots\ldots\ldots\ldots\ldots\ldots (33)$$

in which: P_w = "wetted perimeter" of the prism; u = fluctuation of longitudinal velocity; and v = velocity fluctuation perpendicular to the perimeter. (For each point of the wetted perimeter the average of $\bar{u} \, \bar{v}$ for a long period of time must be computed; the quantity in Equation (33) is the average of these averages over the entire wetted perimeter.)

In the light of the remarks at the beginning of this discussion, it is known that Equation (33) cannot possibly be strictly correct for rough channels, because of the deviation of the average velocity vectors from straight parallel lines. For very large roughnesses this deviation may be quite considerable. It can scarcely be doubted, however, that the formula remains correct in so far as orders of magnitude are concerned. (Otherwise the numerous applications of this formula to rivers and other considerably rough conduits could not have led to favorable results.) Therefore, accepting the formula and comparing it with the formula of the highly effective energy dissipators discussed, one finds that in the latter the product of the velocity fluctuations is replaced by that of actual average velocities. On the other hand the entire wetted perimeter P_w of the prism is replaced by the expression $\frac{n f_s}{b_0 + b_w} (\sin \alpha_1 - \sin \alpha_2)$ which obviously

corresponds only to a moderate part of the wetted perimeter, in accordance with the fact that the generation of turbulence eddies in such energy dissipators is restricted to part of the perimeter. However, this is a factor of lesser order of magnitude than the replacement of a fluctuation-product-average by a product of two average velocities. Thus, it is readily understood from the comparison of the two different mechanisms of momentum transfer that even the simplest of these energy dissipator designs (Fig. 12(*a*)) makes possible slopes, for a given limited average velocity, about 8 to 10 times larger than perpendicular obstacles of the same height with their optimum spacing would permit.

The results of intense momentum transfer produced by the energy dissipators discussed can well be compared to a shock or impact, in contrast to an abruptly expanding conduit, for which—as the author rightly points out—the expression "shock loss" is quite unsuited. Therefore, where the secondary current strikes the main flow, immediately intense but fairly fine-scale turbulence eddies are produced, without the intermediate stage of large-scale eddies. One could be tempted to speak of a large-scale regular vorticity with longitudinal axis; but it is doubtful whether this point of view would be either adequate or fruitful because the path of any individual particle is composed of two quite different kinds of paths alternating with each other in the manner indicated schematically in Fig. 14. It appears that no system of reference in uniform translation movement can be defined with respect to which the movement of the particles would have a closed orbit, and therefore the concept of vorticity is not strictly applicable to the phenomenon as a whole. Furthermore, the high rate of energy dissipation could not be explained by the presence of a vorticity with longitudinal axis, since it is known from other investigations that vorticity of this kind does not dissipate energy of any amount worth mentioning. For example, it is known that the secondary currents, assumedly connected with turbulent flow in a straight, smooth conduit of non-circular section, do not dissipate energy to any considerable extent.

Much more helpful in the understanding of the highly efficient roughness types discussed herein is the concept of "discontinuity surfaces." To be sure, discontinuity of velocity distribution in the strict sense of the word is impossible in any steady phenomenon. Suppose, for example, that a drainage channel meets a river at a certain angle, and that there is a sluice at the channel's mouth. In the instant when the sluice is lifted there is a genuine discontinuity between the velocity distribution in the channel and in the river, and in Fig. 15 line $A\ B$ represents the surface of discontinuity. Quickly upon the generation of eddies, however, and their migration into the main stream, a kind of unity between the two velocity fields is established, no surface of discontinuity (in the strict sense of the word) being present; yet in a more or less narrow strip —tentatively indicated in Fig. 15 by shading—the transition between the

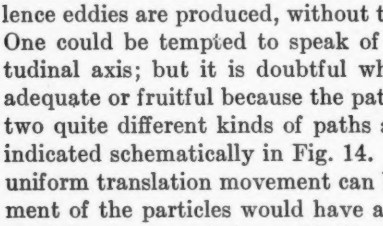


FIG. 15

velocity distribution of the channel and the river is particularly steep. Such regions of steep transition between two distinct velocity distributions can be represented schematically by a surface or, in the section, by a line of discontinuity; and it is in this sense that "discontinuity surfaces" are discussed in theoretical fluid mechanics.

There are obviously four extreme possibilities for the nature of such discontinuities, and the writer has represented them in Fig. 16 (the dots indicate

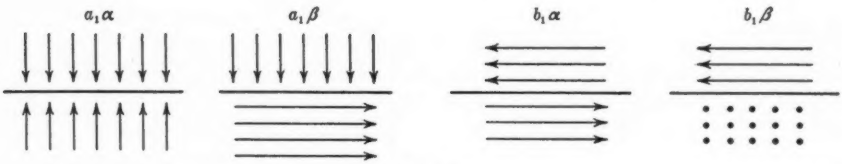


FIG. 16

streamlines perpendicular to the plane of the drawing). Actually any superposition of these cases can occur, but it is valuable to distinguish the distinct fundamental possibilities.

Where the secondary current "strikes" the main flow, obviously, case $a_1\,\beta$ is realized with very good approximation. The foregoing momentum transfer studies explain the superior energy dissipating ability of this kind of discontinuity. The writer has made a little experiment with the channel in Fig. 12(*c*) to compare "discontinuity" $a_1\,\beta$ with $b_1\,\beta$. Obviously, in the middle section of each secondary current, its relation to the main current is very close to that in $b_1\,\beta$. Therefore, if this part of the baffles is covered by a longitudinal strip of thin steel sheet as indicated by a broken line in Fig. 12(*c*), the friction term $F_{t,c}$ is replaced by the known and extremely small friction of water upon a smooth steel surface. The comparison has shown that this does not, to any considerable extent, alter flow conditions in the main channel, thus indicating the complete lack of significance of the term $F_{t,c}$ compared with the total force, or with F_n which constitutes its main part. Similar relations can be expected to exist also between the energy conversions corresponding to these various forces. There are indirect reasons for believing also that "discontinuity" $b_1\,\alpha$ is a rather poor energy converter compared with $a_1\,\beta$. Indeed if one looks once more upon the ordinary roughness in its optimum spacing (Fig. 11(γ)), one will find that between the inner flow and the flow within the lee region of the obstacles a steep transition is occurring which can be represented schematically by a discontinuity surface of type $b_1\,\alpha$; and one can conclude from the comparatively moderate efficiency of this type of roughness that this type of "discontinuity surface" is not highly effective as a dissipator of energy.

The writer intends also to examine case $a_1\,\alpha$ by a comparative study of conduit design Fig. 12(*b*) with and without the four straight, longitudinal walls. Obviously these prevent occurrence of "discontinuity" of the kind $a_1\,\alpha$, thus securing secondary currents of greater intensity at the reissue into the main flow. These reissuing jets being of decisive importance for the energy

dissipation in the conduit as a whole, the amount of increase in the dissipation caused by the walls will give an indication of the importance of the energy-dissipating effect of a discontinuity of type $a_1\,\alpha$.

It appears on the whole that the concept of "discontinuity surfaces," although an articifial abstraction, is, along with the general concept of momentum transfer, a fruitful aid in the analysis and understanding of the varied phenomena of energy conversion in conduits.

As to problem (*b*), that of destroying the large forward velocities present at the foot of an overfall dam or a chute, the writer fully agrees with Professor Kalinske's remark that, for security against scour or other destruction, transformation of forward flow into large-scale eddies or vortices is not sufficient and that disintegration into a finer-scale turbulence is indispensable. The writer believes, however, that the means to this end will have to be investigated experimentally and that it must be discussed in terms of general momentum transfer and "discontinuity" concepts. Although there are many valuable hydraulic research studies for special structures for scour prevention at the foot of dams, it appears that no general investigation of the problem from this fundamental point of view has been attempted thus far.

The difficulties of the problem are great and manifold. If compared with problem (*a*), discussed previously herein, two additional difficulties will be noticed immediately: First, that problem (*b*) is one of essentially non-uniform flow; and, second, that high-velocity shooting flow must be dealt with. Every experimenter knows the "evasive" character of such flow. Coming from a reservoir or river, the flow in problem (*a*) is relatively slow and can be easily deflected into the secondary channels; water shooting at high velocity is much more difficult to divert.

Nevertheless the writer believes that some of the considerations outlined in this discussion, concerning case (*a*), can well be utilized in case (*b*).

From an excellent study by Schoklitsch[51] it is known that the phenomenon of a "hydraulic roller" connected with the hydraulic jump should be considered as a backward flow of the sluggish water on top of the forward shooting flow, with a "discontinuity surface" between, rather than a single large vortex. From the foregoing comparative discussion it is known, however, that this particular kind of "discontinuity surface" is not one of very high energy dissipating efficiency, and this indicates that future studies may yield very substantial progress by revealing facts concerning "discontinuity" of the kind $a_1\,\beta$ shown in Fig. 16. If the writer understands its function, the "jet breaker" ("stralkyvare") suggested by Erik Lindquist,[52] M. Am. Soc. C. E., appears to be an energy dissipator that produces a similar effect.

However, when considering the problem of destroying large forward velocities, it should be kept in mind that the task is not always best solved by dissipating energy at a particularly high rate—that is, by maximum velocity destruction per unit length of structure. It may be more advisable, in many cases, to dissipate the energy at a somewhat more moderate rate, if it can be located conveniently. Therefore it is not impossible that the structures

[51] "Über die Energie-vernichtung durch Walzen," by A. Schoklitsch, *Die Wasserwirtschaft*, 1932.

[52] "Energyomvandling ved footen af overfall dammar," by Erik Lindquist, Stockholm.

generating an ordinary "hydraulic roller" will be used also in the future since they conveniently solve problem (*b*) by placing the dangerous vorticity, issuing from the discontinuity surface, well above the bed surface.

The writer believes that the most important field for the civil engineering applications of turbulence statistics lies in the problem of sediment transportation in canals and rivers. An entire series of Swiss investigations by aid of the bed-load trap[53] as well as the combined theoretical and experimental research of H. A. Einstein and G. Polya[54] seems to indicate that even the movement of the heavy bed load is essentially connected with the haphazard fluctuation-phenomena characteristic of turbulence. Nevertheless, it would be a fallacy to conclude that the problem of propelling granular materials by fluids can be treated, as a whole, as a problem of turbulence statistics, in the sense that Professor Kalinske makes use of this method. Paradoxically, although the bed-load movement obviously derives its energy from, and shares much of the properties of, the turbulence movement in the higher layers of the rivers, it also shows regular rhythms in many instances (such as shown not only by sand waves, but also, for long periods, by the generation of serpentines). This paradox will probably not find its complete clarification before turbulence will be understood in its relation to regular rhythmic vortex distributions on one hand, to momentum transfer by "grain bombardment" on the other. (The vortex theory of sand waves was first presented by G. Darwin and later developed by Vaughan Cornish[55] and the Swedish School of Geomorphologists.[56] The impact theory of sand waves has been presented by R. A. Bagnold in a long series of publications;[57] although the decisive importance of this factor is limited to Æolian phenomena [desert sand, etc.] it can scarcely be doubted that it must be included among the factors governing fluvial sand propulsion too.) With reference to the river problem the author's inquiries into open channel turbulence must be considered a great step forward to a far-off but very worth-while goal. The methodic aspect of the problem as a whole has been analyzed by the writer in a recent paper.[58]

Finally, a few words in regard to the problems of gathering experimental data for the statistical study of turbulence: This question consists really of two parts—(1) how to make fluid particles visible, and (2) how to record the movement of particles made visible. To problem (1) the author has contributed a highly reliable technique. It should be stated that the ingenious ultramicroscopic method of A. Fage and H. C. H. Townend[32] does not require

[53] See "Untersuchangen in der Natur über Bettbildung, Geschiebe- und Schwebestofführung," Mitteilung Nr. 33, Eidgenössisches Amt für Wasserwirtschaft, Bern, 1939.

[54] "Der Geschiebetrieb als Wahrscheinlichkeitsproblem," by H. A. Einstein; and "Zur Kinematik der Geschiebebewegung," by G. Polya, Rascher, Zürich, 1937.

[55] "Waves in Sand and Snow and the Eddies Which Make Them," by V. Cornish, London, 1914; also "Ocean Waves and Kindred Geophysical Phenomena," by V. Cornish, London, 1935.

[56] See, for example, "Beitrag zur Kenntnis der Transportmechanik des Geschiebes und der Laufentwicklung des Reifen Flusses," by H. W. Ahlmann. (Sveriges Geologiska Undersokning, Ser. C., No. 262, Stockholm, 1914.)

[57] See, for example, "The Transport of Sand by Wind," by A. Bagnold, *Geographical Journal* 99, No. 5, 1937.

[58] "The different approaches to the study of propulsion of granular materials and the value of their coordination," by P. Nemenyi, *Proceedings*, Round-Table Conference on the Rôle of Hydraulic Laboratories in Geophysical Research, Washington, D. C., 1939; publication pending.

[32] "An Examination of Turbulent Flow with an Ultramicroscope," by A. Fage and H. C. H. Townend, *Proceedings*, Royal Soc. of London, Vol. 135A, 1932, p. 656.

the addition of any strange material, the very fine suspensions present in drinking water being sufficient in many cases; for macroscopic visibility of air particles Mr. Townend[59] has suggested that it can be attained without strange admixture, but by localized heating through electric flashes combined with the "Schlieren technik"; the method is very interesting although it is rather unlikely to have any direct applicability for water.

The second half of the experimental problem consists of how to record the movement of particles made visible. The writer would like to call attention to L. Prandtl's suggestion[60] that a uniformly moving camera be used with speeds of systematically varied magnitudes. Mr. Prandtl showed how photographs taken with this method can lead to a very convenient system of velocity-fluctuation statistics. The writer should like to add the suggestion that such photographic records can be used for measuring vortex sizes. The camera moving at the speed corresponding to the average velocity of a certain longitudinal strip of the channel, the vortices in this particular strip will appear in their real size. The vortex size distribution in each strip found by this direct method should be compared with the corresponding average vortex sizes. These can be computed according to Mr. Taylor's suggestion[6] from the velocity data, as shown for the open channel by the author.

Bennie N. Netzer,[61] Jun. Am. Soc. C. E. (by letter).[61a]—The practical hydraulic engineer's need has been for someone to put into actuality Mr. Kalinske's statement (see heading "Transportation of Suspended Material") that "Turbulence should be thought of in terms of definite parameters instead of just as a qualitative descriptive term relating to a general condition of flow." Many hydraulic model studies have been made of problems involving energy dissipation, for example, which have had as their net result only comparative indications of the intensity of turbulence, but no quantitative conclusions The reason for this has been the lack of an instrument for measuring, accurately, the highly fluctuating velocities in turbulent flow.

For a thorough understanding of the turbulence mechanism it is undoubtedly necessary to use some such method as the photographic method suggested by Mr. Kalinske. For quick, practical results, however, some other method that is less time-consuming and tedious and has more flexibility than the photographic method seems desirable. It is difficult to see how the photographic method can be used to measure velocities throughout an entire turbulent region with any ease and without consuming too much time. Model studies involving the analysis of turbulent flow in many hydraulic designs require a quick, practical way for measuring velocities.

Mr. Kalinske states (see heading, "Experimental Techniques") that "An instrument that would give accurate recordings of the instantaneous velocity in a direction either longitudinal or transverse to the flow would be useful."

[59] *Proceedings*, Royal Soc. of London, Vol. 145A, 1934, p. 80.

[60] "Über ausgeildete Turbulenz," by L. Prandtl, *Proceedings*, 2d International Cong. for Applied Mechanics, Zürich, 1927, p. 62.

[6] "Statistical Theory of Turbulence," by G. I. Taylor, *Proceedings*, Royal Soc. of London, Vol. 151A, 1935, p. 421.

[61] Asst. Engr., U. S. Engr. Office, Pittsburgh, Pa.

[61a] Received by the Secretary March 29, 1940.

For the determination of the intensity of turbulence in a practical manner such an instrument is necessary.

Fig. 17 shows an instrument that may fulfil Professor Kalinske's requirements. The instrument was designed by H. A. Thomas, M. Am. Soc. C. E., and was used by the writer in experimental work on erosion below dams.[62] The instrument is a helical spring dynamometer that is very sensitive to turbulence and will measure the instantaneous maximum, mean, or minimum velocities at any point in a region of turbulent flow. The meter is essentially a vertical rod, encased in a streamlined tube, with a slightly cupped disk attached to its lower end by a horizontal arm. The torque caused by a current of water impinging on the cupped disk is transmitted by the rod to a horizontal pointer which is restrained in its rotation by two pins. The torque caused by the water current is balanced by a helical spring attached to the pointer and to a slide which moves at right angles to the pointer. When the torque is balanced, the pointer is free of both pins (similar to a scale beam), thus indicating the mean velocity. To ascertain the instantaneous maximum or minimum velocities the spring is stretched until the pointer is just moved from either pin. By suspending the vertical rod by a light thread its weight is removed from the bearings and friction is reduced to an absolute minimum. The instrument is calibrated by measuring the velocity of a jet of water moving with uniform velocity, and then determining the elongation of the helical spring in the same jet.

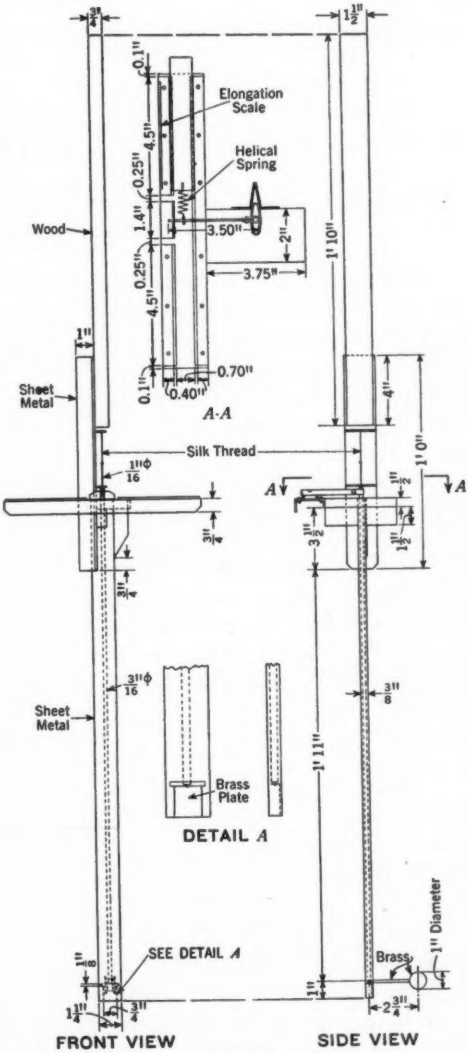


Fig. 17

[62] "Erosion Below Dams," by Bennie Netzer. Thesis presented to Carnegie Inst. of Tech., in 1938, in partial fulfilment of the requirements for the degree of Master of Science in Civil Engineering.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FOUNDATION EXPERIENCES, TENNESSEE VALLEY AUTHORITY
A SYMPOSIUM

Editors' Note.—Comments from more than one direction have called attention to the omission of the name of Arthur E. Morgan from the list of those having responsibility for the work covered by this series of papers. To make the reference complete, the acknowledgments in the "Foreword" of the Symposium are therefore being revised for *Transactions* to read:

Personnel.—The projects were built with the Authority's forces. From the time TVA was authorized in 1933 Arthur E. Morgan, M. Am. Soc. C. E., was chief engineer, until he relinquished that position June 17, 1937. At the beginning of the work described in this Symposium, C. A. Bock, M. Am. Soc. C. E. (chief consulting engineer May 1, 1938, to November 30, 1939), was assistant chief engineer; T. B. Parker, M. Am. Soc. C. E., joined the staff in November, 1935, as chief construction engineer, and became chief engineer in May, 1938. He was succeeded as chief construction engineer by A. L. Pauls, M. Am. Soc. C. E.; Edwin C. Eckel, Affiliate Am. Soc. C. E., was chief geologist; and Berlen C. Moneymaker, Assoc. M. Am. Soc. C. E., assistant chief geologist.

Discussion

By Messrs. George K. Leonard, and F. B. Marsh

George K. Leonard,[19] M. Am. Soc. C. E. (by letter).[19a]—The foundation conditions at Guntersville Dam, and the treatment of the rock to make it watertight, have been described accurately and completely in the papers by Mr. Gongwer and Mr. Ross. Few dam foundations in limestone are found that are better than that under Guntersville Dam, unless one can imagine finding limestone without solution channels. The engineers of the U. S. War Depart-

Note.—This Symposium was published in March, 1940, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the Symposium.

[19] Constr. Engr., TVA, Watts Bar Dam, Tenn.

[19a] Received by the Secretary April 1, 1940.

ment who did the first drilling on the site in 1914, and the TVA engineers who completed it and made the final decision on the dam location, are to be complimented.

Although the exact location of the dam was not dictated by the location and height of Wheeler Dam (the next dam downstream), the general location was fixed within the limits of several miles. Within these limits several dam sites could have been found; and, in fact, several were investigated which could have been developed satisfactorily.

As has been described, the bedding of the limestone was tight and of ample thickness. It was nearly horizontal but what slight dip there was, was in the right direction (upstream). In the entire length of 4,000 ft, only two areas totaling about 600 ft in length showed signs of being cavernous or unsound; and these areas were fairly close to the surface.

The writer has often wondered whether the locating engineers knew exactly how good a site they were choosing or whether the significance of the formation that made it a supersite was unknown to them. Even if the latter was the case, they still deserve much credit for their excellent choice. It would have been a good site without the super feature and by reason of its presence few better limestone foundations can be imagined.

The feature that changed the site from an excellent to a nearly perfect one is the continuous, insoluble, shale horizon which underlies the entire site. After the engineers had fully realized its significance, no further uneasiness was felt about the foundation. They knew that below the shale no solution had taken place and that the rock must be sound and tight. Above the shale they knew that they might find anything that is usually found in limestone bedding; and, indeed, in one place or another, throughout the length of the dam, they actually found about everything.

The shale seam outcropped a short distance below the dam, and it is possible that the relocation of the dam downstream to a point nearer the outcropping would have found the shale seam still closer to the surface, thus saving some excavation and further improving the foundation. Although hindsight is always better than foresight, this possibility cannot be stated as a certainty.

The shale along the axis varied in depth from 15 to only 25 ft, so the cost would not have been excessive even had it been found necessary to make a concrete cutoff through the uncertain rock above the shale for the entire length of the dam. Had this been done, grouting might have been dispensed with in the rock below the shale, with perfect safety, although this would have been contrary to orthodox engineering practice. Since the rock above the shale was fairly sound in most places, the economical method of treatment consisted in general of removing all disintegrated rock and saving as much of the sound rock as possible, treating it in various ways to consolidate it.

Grouting procedure was not difficult or unique; but the search for, and filling of, caves under the south embankment disclosed many weird geologic formations and presented a series of problems which Mr. Gongwer has described. The control of underground water in the confined space between the sheet-pile walls was difficult, but in no place except the final closure was concrete placed in water. It is believed that the cofferdam method of opening up the cutoff is

superior to the open-cut method, not alone because the cost is less but because the contact of the cutoff piling with the rock or concrete core wall can be seen before backfilling the trench. The open-cut must be backfilled before driving the piling, and it is impossible to see the seating of the piles on the uneven rock. Some engineers might think the piles were unnecessary if the trench was backfilled with impervious clay. This might be true, but it would depend upon the nature of the original overburden. If this was fairly pervious, the clay core at the bottom might be too thin. Of course, a level concrete foundation could be placed on the rock on which to seat the piles.

Grouting of the disintegrated rock under the cofferdam cells is believed to be a new technique for cofferdam construction. Cellular type cofferdams can be built safely on solid rock or on soft homogeneous material such as sand. In the latter, seepage is high and berms must be wide enough to prevent piping. Systems of well points and large pumping capacity are necessary to keep the excavation dry.

On rock bottoms it is customary to build the cofferdam as close to the line of excavation as working conditions and plant layout will permit. If open or gravel-filled solution channels or mud seams are exposed on the face of the cut, trouble begins immediately. This condition developed in the lock cofferdam and caused considerable expense and worry. A construction man will fight a leak to the bitter end with everything at his command rather than allow the dam to flood after it is once unwatered. This is what the construction forces did, but they finally lost. Looking backward again it would have been much cheaper, and no more time would have been consumed, if the dam had been flooded when the first sign of disintegrated rock appeared and if it had then been grouted under equalized head.

The experience gained in the lock cofferdam was used to advantage in the spillway and power-house cofferdams. Here, where the piling in the cells landed on unsound rock, grouting was completed before unwatering. In each of these two large areas of about 6 and 9 acres, respectively, the seepage was not enough to tax a 6-in. pump. The cofferdam grouting gave the work all the advantages of a dry-land location. Certainly no more justification is necessary.

The importance of visual inspection of foundation rock, in cored holes large enough for the inspector to be lowered into them, cannot be overemphasized. Logs of, and cores from, small cored holes tell many things; but they do not tell enough, and are likely to lead to erroneous deductions and conclusions. Logs of holes made with percussion drills are almost worthless unless the rock is either very poor or very good.

How much preliminary exploratory drilling should be done before construction starts? Obviously, this depends on the kind of rock, its condition, the information desired, and many other things; but in general, the writer doubts the economy of attempting to show all subsurface conditions completely with preliminary drilling. Only general conditions should be investigated, leaving the gaps to be filled in by the construction forces as the work progresses. Contract work and force account work would require different plans for explora-

tion. Limestone like that at Guntersville, and breccia like that at Boulder Dam, would require different investigation.

One can well imagine the difficulty of making a complete preliminary investigation at either the Chickamauga or Guntersville sites upon which a contractor could safely bid and about which there would be no argument; yet, when done by force account, the corrective treatment has been expeditious and complete.

F. B. Marsh,[20] M. Am. Soc. C. E. (by letter).[20a]—The experiences described in this Symposium are particularly interesting to any one who has tried to shut off underground flow under a dam or other structure. The work accomplished by the TVA in this direction seems to have been exceptionally thorough and effective.

Nevertheless the question inevitably arises whether a dam founded on a soluble limestone can ever be made entirely safe as a long-term proposition. This is especially true when one considers the normal tendency of maintenance organizations to confine their attention, after construction organizations are gone, to strictly necessary routine matters and to give no thought to gradual changes that may be occurring, such (for example) as the slow development of springs downstream from a dam or from a low divide near the rim of a reservoir.

Even if the dissolving action may be very slow, as noted by Mr. Fox, the ultimate effectiveness of ground water as a solvent of limestone is clearly demonstrated by the conditions in this Tennessee valley, not to mention the cave regions in the Shenandoah Valley of Virginia and elsewhere. It is not difficult to imagine that a leak, now so small as to be unnoticed, in a long term of years may develop into something dangerous, regardless of the cutoffs so painstakingly provided, as described in this Symposium.

Such considerations make it advisable to sound a warning to other less careful and perhaps less permanent organizations than TVA, which may be tempted to build dams on limestone foundations, that they are treading on dangerous ground—on ground that can be made safe only at great expense. One of the writer's recollections of younger days is the acrimonious discussion that followed the failure of the Colorado River dam near Austin, Tex., in 1893, after only eight years of service, the limestone foundation being one of the factors involved.[21]

---

[20] Div. Engr., Eastern Dept., New York Board of Water Supply, White Plains, N. Y.

[20a] Received by the Secretary March 29, 1940.

[21] *Engineering News*, Vol. 29, June 8, 1893, p. 545, and June 29, 1893, p. 618; also, *loc. cit.*, Vol. 30, July 27, 1893, p. 78.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EFFECTIVE MOMENT OF INERTIA OF A RIVETED PLATE GIRDER

Discussion

By Messrs. J. R. Shank, and Harold D. Hussey

J. R. Shank,[34] M. Am. Soc. C. E. (by letter).[34a]—Few engineers would think of using the net-section moment of inertia of a plate girder for computing the deflection for the reason that it is representative of so little of the length. The gross-section moment of inertia represents much more of it. A moment of inertia proportioned between these two is more likely than either of them to be the proper one to use. The deflection data of this paper afford excellent material for making a study of the proper basis for this proportioning. If the addition of holes in the tension flange causes an increase in the deflection, it is apparent that the angular movement due to the bending of that part at the holes is no longer the same as if no holes were present and the use of the gross moment of inertia is therefore only an approximation. The deflection may be computed in detail for gross section and net section progressively, as they come, or fairly accurately from gross and net sections in proportion to the parts of the lengths taken by the two. If the deflections given in this paper are equal to the deflections so computed, the conclusion must be drawn that the angular movement as computed from the net moment of inertia obtains at the regions containing holes and the fiber deformations and stresses are different from those at no holes.

Table 10 is an attempt to show the effect of the holes on the reduction of the load-carrying characteristics as illustrated by the moments of inertia. For a given deflection the moments of inertia are directly proportional to the loads carried. Columns (1) and (5) of Table 10 are taken directly from Columns (3) and (1) of Table 3. The percentage reductions are all computed

Note.—This paper by Scott B. Lilly, M. Am. Soc. C. E., and Samuel T. Carpenter, Assoc. M. Am. Soc. C. E., was published in October, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: December, 1939, by Messrs. William R. Osgood, Clyde T. Morris, B. R. Leffler, E. Neil W. Lane, Lewis E. Moore, W. E. Black, and L. E. Grinter; January, 1940, by Messrs. Charles M. Spofford, E. Mirabelli, Edward Godfrey, Walter H. Weiskopf, and C. D. Williams; February, 1940, by Messrs. Alvin B. Auerbach, Jonathan Jones, and R. L. Moore; and March, 1940, by Messrs. J. M. Garrelts and I. E. Madsen, Francis P. Witmer, and A. W. Fischer.

[34] Prof., Civ. Eng., and Asst. Director, Eng. Experiment Station, Ohio State Univ., Columbus, Ohio.

[34a] Received by the Secretary March 25, 1940.

from the A-value for each group. Column (3) is not found in the original paper. For PT-1 specimens having $\frac{11}{16}$-in. holes, spaced 5 in. apart, 13.7% of the entire length is at holes and 86.3% at no holes, or at gross section. Accordingly it is assumed that 86.3% of the girder has a moment of inertia of, say, 178.5 for specimens without cover plates, and 13.7% has a moment of inertia of 164.3. For convenience, the holes are considered to be $\frac{11}{16}$ in. square; thus:

TABLE 10.—MOMENTS OF INERTIA AND PERCENTAGE REDUCTION DUE TO HOLES OPEN (B), WITH BOLTS (C), OR WITH RIVETS (D)

Test section	(a) SPECIMEN PT-1						(b) SPECIMEN PT-2					
	Net Section		Proportional		Observed		Net Section		Proportional		Observed	
	Moment of inertia, I (1)	Percentage of section A (2)	Moment of inertia, I (3)	Percentage of section A (4)	Moment of inertia, I (5)	Percentage of section A (6)	Moment of inertia, I (1)	Percentage of section A (2)	Moment of inertia, I (3)	Percentage of section A (4)	Moment of inertia, I (5)	Percentage of section A (6)
A-1	178.5	....	178.5	....	189.0	....	178.5	....	178.5	....	189.0	....
B-1	164.3	8.0	176.5	1.1	182.0	3.7	164.3	8.0	174.6	2.2	177.0	6.4
C-1	164.3	8.0	176.5	1.1	187.0	1.1	164.3	8.0	174.6	2.2	179.0	5.3
A-2	300.9	....	300.9	....	315.0	....	300.9	....	300.9	....	315.0	....
B-2	270.7	10.0	297	1.3	298.5	5.2	270.7	10.0	293	2.6	282.0	10.5
C-2	270.7	10.0	297	1.3	306.5	2.7	270.7	10.0	293	2.6	295.0	6.4
A-3	476.9	....	476.9	....	494.0	....	480.1	....	480.1	....	500.0	....
B-3	430.0	9.8	471	1.2	460.5	6.8	433.2	9.8	467	2.7	442.0	11.6
C-3	430.0	9.8	471	1.2	472.0	4.5	433.2	9.8	467	2.7	453.0	9.4
D-3	430.0	9.8	471	1.2	480.5	2.7	433.2	9.8	467	2.7	460.0	8.0

$$178.5 \times 0.863 = 154.0$$
$$164.3 \times \frac{0.137}{1.000} + \frac{22.5}{176.5}$$

The effective moment of inertia for computing deflection or the proportional moment of inertia, as designated herein, is then 176.5. This procedure is substantially the same as that which would be used for computing the deflection of a girder having two different moments of inertia when the number of alternations of the two are allowed to increase toward infinity.

The percentage reductions shown in Column (2), Table 10, are those which would be expected if the holes ran in the form of slots for the full length of the girder. If the holes had no effect on the deflection, these percentages would all be zero. There are the two extremes. If the stresses at the holes are what would be expected from the net-section calculations, the reductions would be as shown in Column (4), Table 10. Column (6) shows the percentage reductions for the observed moments of inertia. The moments of inertia shown in Column (5) are larger than those in Column (1), and a reference to the paper shows them to be generally somewhat smaller than the gross section values of Column (2), Table 3. These differences are immaterial inasmuch as the comparisons are by percentages.

The material in the number 1 group, A-1, B-1, and C-1, is the most valuable since the A-values were not taken from girders where clamps were relied upon to develop the horizontal shear as was the case for the others. In all cases except one the effect of the holes was greater than would be expected from ordinary net-section calculations, taking into account the proportionate length occupied by the reduced section. The stiffening effects due to bolts and rivets are shown clearly. The discrepancies between the values of Columns (6) and (4), Table 10, are greater for the closely spaced holes. It appears that the holes have an effect over a length greater than their diameters and that the substitution of square holes for round holes is not nearly enough to cover this condition. In the cases of the close spacings these effects apparently met or overlapped.

The conclusion seems to be clear that the net-section method for the calculation of tensile fiber stresses indicates the actual situation much more accurately than the gross-section moment of inertia; and if designing plate girders by the gross-section moment of inertia is to be continued, it should be recognized clearly that considerably greater stresses occur at the rivet holes in the tension flange than the calculations indicate and that this fact must be taken into account when specifying allowable unit stresses.

HAROLD D. HUSSEY,[35] M. AM. SOC. C. E. (by letter).[35a]—The need of more tests of plate girders is emphasized in this paper. The tests reported were planned with care and were skilfully executed, and the writer believes that the authors' conclusions will be verified by further experimental work.

Under "Flange Stress" the authors state that the stress in the compression flange is greatest for open holes, and that it gradually approaches the computed value as the method of holding the flange together is changed from clamps, to bolts, and to rivets. The authors add that a corresponding decrease is not shown for tension flange, indicating that bolts and rivets tend to relieve the stress in the compression flange. The observed results in Column (6), Table 3, indicate that bolts and rivets tend to relieve the stress in the tension as well as the compression flange. This seems to offer evidence that the gripping action of rivets produces an appreciable reduction in the theoretical stress concentration at the sides of open holes in the flanges. Such stress concentrations may be as much as three times the gross unit stress, which indicates that they will reach the yield point of the material before the working stress is developed. It is significant that when the two girders were tested to 30 kips per sq in. there was no indication of permanent set. This is contrary to what one would expect. The fact that no set was produced is evidence of the relieving action of rivets.

The most important conclusion reached by the authors is that plate girders may be designed acceptably by the use of the gross moment of inertia. The most common consideration of this question has been on the basis of gross sections versus net sections of a tension member, when it should have been directed to the action of the plate girder.

---

[35] Designing Engr., Am. Bridge Co., New York, N. Y.

[35a] Received by the Secretary April 1, 1940.

A plate girder has one flange in tension and one flange in compression. The strength of the compression flange is determined by its resistance to buckling. It has long been known that, under static loads, the compression flange of a symmetrical girder will fail long before the tension flange has reached its ultimate strength. It has also long been known that, under static loads, it is practically impossible to make a girder fail in the tension flange, while the compression flange is still intact. This was well demonstrated in tests of plate girders which are summarized in Table 11.

These tests were made at the Ambridge, Pa., plant of the American Bridge Company in 1910. The girders were $30\frac{1}{2}$ in. deep, back to back of flange angles; 28 ft long, center to center of bearings; and were made in the following general classes: Class A girders were symmetrical, without cover plates; class B girders were unsymmetrical, without cover plates, the compression flanges being heavier than the tension flanges; class C girders were symmetrical, with cover plates; class D girders were unsymmetrical, with cover plates, the compression flanges being heavier than the tension flanges; and the two class E girders were extremely unbalanced, with two angles only in the tension flanges and cover-plated compression flanges which were more than twice as heavy as the tension flanges.

Load was applied to the girders at the third points of the span, through bearing blocks on the compression flanges. The two ends of the girders were held against lateral movement. The unsupported length of the compression flanges, therefore, was equal to the length of the girders except for such support as was supplied at the load points.

All of these girders failed in the same manner—that is, by buckling of the compression flanges at theoretical unit stresses which averaged 9% above the yield point. This was accompanied, in some cases, by buckling of the web plate. In only one case (Girder G11) was there failure of the tension flange. It is evident that the tension and compression unit stresses should be alike for symmetrical girders, when they are based on gross moment of inertia. The tension stress, on the basis of net moment of inertia, was somewhat higher (10% for the girders without cover plates and 20% for those with cover plates). The maximum tension stresses were still less than the ultimate—in the symmetrical girders these stresses varied from 73% to 81% of the ultimate.

Making the girders unsymmetrical resulted in an increase in the tension unit stress at failure without any appreciable change in the compression unit stress (in comparison with the yield point). These girders continued to fail by buckling of the compression flange or failure of the web plate until the dissymmetry reached the extreme values shown in girders G11 and G13, where the compression flanges were more than twice as heavy as the tension flanges. Girder G11 showed a maximum net tension stress of 105% of the ultimate tensile strength. As stated, this was the only girder in the series that was reported to have shown signs of failure in the tension flange. The report of this failure states, "Top (compression) flange and cover plates buckled and deflected downward. Complete failure of top and bottom flanges and web

between loads."[36] Even in this girder the compression unit stress at failure was more than the yield point of the material.

TABLE 11.—TESTS OF PLATE GIRDERS, 30½ IN. DEEP AND 28 FT LONG

Girder mark	Thickness* of web plate, in inches	Compression Flange: Thickness, in inches: Cover plates†	Compression Flange: Thickness, in inches: Angles‖	Compression Flange: Area, in square inches	Tension Flange: Thickness, in inches: Cover plates†	Tension Flange: Thickness, in inches: Angles‖	Tension Flange: Area, in square inches	Load at failure, in kips	Unit Stresses, in Kips per Square Inch: Theoretical Maximum: Compression in flanges (I_g)¶	Unit Stresses: Theoretical Maximum: Tension in Flange: (I'_g)¶	Unit Stresses: Theoretical Maximum: Tension in Flange: (I_n)**	Unit Stresses: Physical Properties: Compression flange (s_y)††	Unit Stresses: Physical Properties: Tension flange (s_t)‡‡
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
CLASS A													
G5A	3/8	...	5/8	11.72	...	5/8	11.72	293.0	44.6	44.6	49.3	42.0	67.4
G5B	3/8	...	5/8	11.72	...	5/8	11.72	318.0	48.4	48.4	53.5	42.0	67.4
CLASS B													
G7	5/16	...	5/8	11.72	...	1/2	9.50	288.0	46.0	52.5	57.8	42.1	68.0
G8	3/8	...	5/8	11.72	...	1/2	9.50	290.5	45.2	51.3	56.7	42.1	68.0
G12	3/8	...	3/4	13.88	...	1/2	9.50	308.0	42.4	53.5	59.0	38.8	61.4
CLASS C													
G4A	3/8	1/2	1/2	16.50	1/2	1/2	16.50	396.8	43.2	43.2	52.1	37.3	64.6
G4B	3/8	1/2	1/2	16.50	1/2	1/2	16.50	393.0	42.8	42.8	51.6	37.3	64.6
G3A	1/2	3/4	5/8	22.22	3/4	5/8	22.22	488.0	39.7	39.7	47.7	34.3	62.0
G3B	1/2	3/4	5/8	22.22	3/4	5/8	22.22	463.0	37.7	37.7	45.3	34.3	62.0
CLASS D													
G10	5/16	1/2	7/16	15.36	3/8	7/16	13.61	344.3	41.1	44.8	53.9	35.1	66.2
G9	5/16	1/2	1/2	16.50	3/8	1/2	14.75	378.0	42.5	46.0	55.5	38.1	66.8
G6A	3/8	3/4	5/8	22.22	5/8	5/8	20.47	438.0	37.0	39.2	47.2	39.7	63.4
G6B	3/8	3/4	5/8	22.22	5/8	5/8	20.47	443.0	37.3	39.6	47.7	39.7	63.4
G14	1/2	1‡	3/4	27.88	1/2	1/2	16.50	558.0	39.6	55.5	66.8	36.9	68.3
CLASS E													
G11	3/8	3/8	1/2	14.75	...	3/8	7.22	288.0	38.4	58.5	67.0	33.4	63.9
G13	1/2	1⅛§	3/4	29.63	...	3/4	13.88	543.0	38.4	61.8	70.4	36.7	68.6

* Web plates all 30 in. deep. † Cover plates all 14 in. wide; one plate except as noted. ‡ Two 14-in. by 1/2-in. cover plates. § Two 14-in. cover plates, 5/8 in. and 1/2 in. thick. ‖ A pair of 6-in. by 4-in. angles in each case; outstanding leg, 6 in. ¶ I_g denotes "moment of inertia based on the gross area of both flanges." ** I_n denotes "moment of inertia based on the net area of both flanges." †† s_y denotes "yield-point stress in the compression flange." ‡‡ s_t denotes "tensile strength of the tension flange."

The writer believes that it would be acceptable to design any of the girders listed in Table 11 by the gross moment of inertia method. The use of net moment of inertia seems to be an unnecessary refinement.

The plate girder is one of the most common types of structural members in use today, and its use as the principal member of large structures is increasing.

[36] Unpublished report of tests at the Ambridge (Pa.) plant of the American Bridge Company, 1910.

The structural action of the plate girder is very complex and warrants much more study than has been given to it.

Considerable research has been conducted on many types of structural members, which has added to modern knowledge of their behavior. The column has proved a fruitful field of research since the Quebec Bridge disaster in 1907. Many column tests have been made in connection with the construction of some of the most important bridges. A very important series of tests of riveted tension members was conducted in connection with the building of the San Francisco-Oakland Bay Bridge. Whenever a structure is built which involves a quantity of eyebars, it is customary to specify full-size tests of several bars. The question is: Where can one find a report of tests of large plate girders made in connection with an important project?

The plate girder has not been subjected to any concerted program of research—possibly because no major disaster has occurred from the failure of such a member. The writer believes, however, that there is need for such tests and that as a result engineers would be able to design more intelligently and with greater economy.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

FLOOD-CONTROL METHODS; THEIR PHYSICAL AND ECONOMIC LIMITATIONS

PROGRESS REPORT OF COMMITTEE OF THE HYDRAULICS DIVISION ON FLOOD CONTROL

Discussion

BY LYNN CRANDALL, M. AM. SOC. C. E.

LYNN CRANDALL,[9] M. AM. SOC. C. E. (by letter).[9a]—It is stated without qualification in Paragraphs 26 and 27 of the Committee's Progress Report that flood-control space in multiple-purpose reservoirs should be emptied promptly after each filling so as to be available for use during the next flood. This statement probably is correct as applied to most areas where damaging floods occur but it does not necessarily apply to certain streams draining the higher elevations in the western Rocky Mountains.

On the upper Snake River, for example, the peak floods occur in May or June with the melting of the snow that has accumulated since the previous November. The amount of water to be expected can be predicted within reasonable limits from the snow surveys and past experience, and after this flood due to the melting snow has passed no other of damaging magnitude has ever been recorded until the following year. Under these conditions, at least, a considerable portion of the water caught in the flood-control section of the reservoir can safely be retained and used for irrigation or power production during the ensuing late summer, fall, or winter months prior to the next snow melting period a year later.

NOTE.—The Progress Report of the Committee on Flood Control was published in February, 1940, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the Report.

[9] Snake River Watermaster, Idaho Falls, Idaho.

[9a] Received by the Secretary March 4, 1940.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

EFFECTS OF RIFLING ON FOUR-INCH PIPE TRANSPORTING SOLIDS

Discussion

BY R. Y. NEWELL, JR., ESQ.

R. Y. NEWELL, JR.,[7] ESQ. (by letter).[7a]—In general the writer agrees with the conclusions reached by the author, and in this connection it is interesting to compare results in the laboratory using the 4-in. test pipe with those obtained during actual operation of a dredge having a 32-in. discharge pipe line. Reference is made to preliminary field tests of rifled discharge pipe made during June, 1936, with the dredge *Jadwin*, a 32-in. pipe-line, dust-pan type, hydraulic dredge of the U. S. Engineer District, Memphis, Tenn.

The location, at a point in the Mississippi River a few miles upstream from Memphis, was selected for making the test because of the coarse and heavy characteristics of the sand forming an obstructing bar at that point. A sieve analysis of a representative sample is shown in Table 2.

TABLE 2.—MECHANICAL ANALYSIS OF REPRESENTATIVE SAMPLE OF MATERIAL DREDGED DURING TESTS ON THE DREDGE *Jadwin*

FINE SAND			MEDIUM SAND			COARSE SAND			FINE GRAVEL			MEDIUM GRAVEL		
Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total	Grain Size, in Millimeters		Percentage of total
From	To		From	To		From	To		From	To		From	To	
0.10	0.25	10.0	0.25	0.50	33.0	0.50	1.00	40.0	1.00	2.00	10.0	2.00	10.00	7.0

Two separate floating pipe lines of equal length were prepared for use of the dredge. One was composed of conventional, smooth bore pipe sections and the other was equipped with type 12 rifling, described by the author. The arrange-

NOTE.—This paper by G. W. Howard, Jun. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Fred R. Brown, Jun. Am. Soc. C. E.; and April, 1940, by David L. Neuman, M. Am. Soc. C. E.

[7] Marine Engr., U. S. Maritime Comm., Washington, D. C.

[7a] Received by the Secretary March 21, 1940.

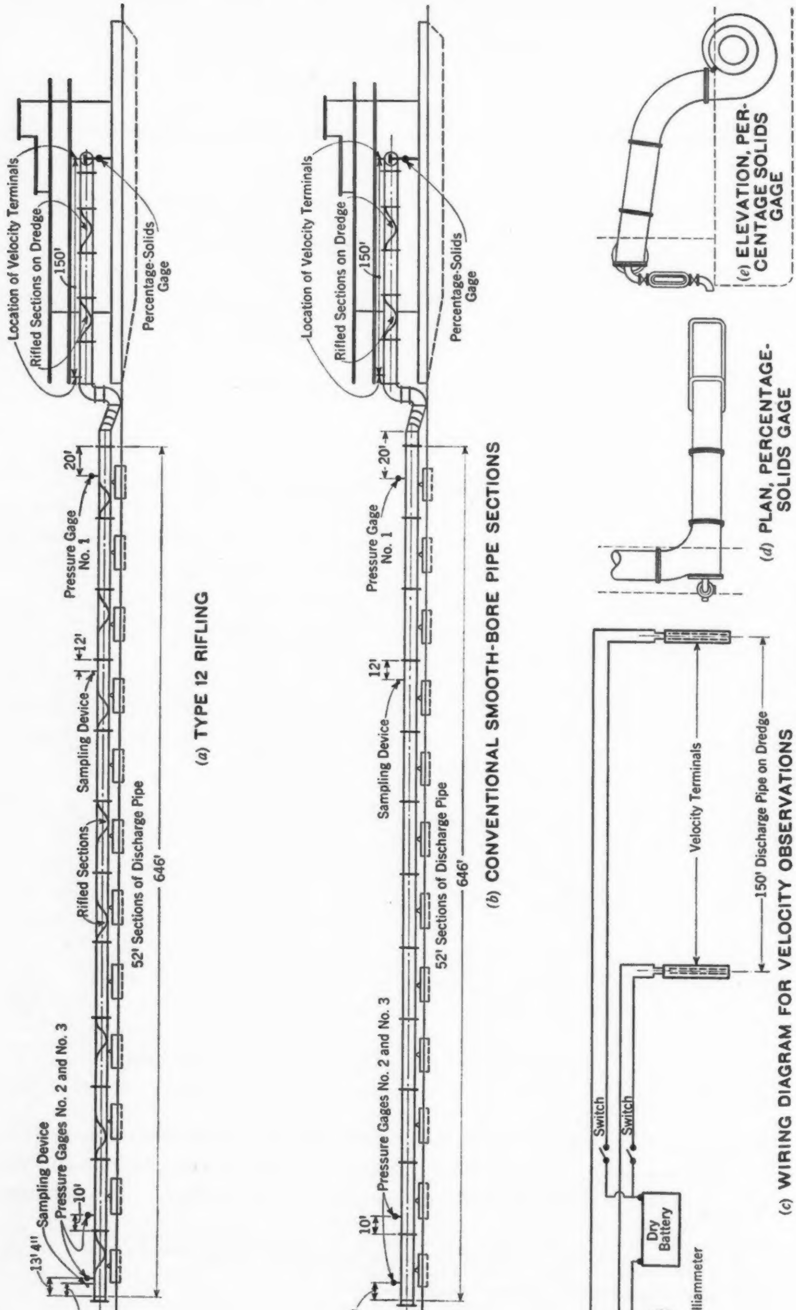


FIG. 13.—PLANT LAYOUT, COMPARATIVE OBSERVATIONS ON RIFLED AND PLAIN PIPE

ment of both pipe lines is shown in Fig. 13. It will be noted that the discharge pipe on board the dredge was also rifled. This proved a boon to consistent velocity determinations. The floating pipe lines were changed after each run, the tests on rifled pipe being followed by comparison tests of plain pipe.

The percentage of solids used in the calculations was obtained with a percentage-solids gage, shown in Fig. 13. The yardage was checked, however, by soundings and rate of advance, as well as by before-and-after dredging surveys conducted as simultaneously as possible.

TABLE 3.—Field Tests of Rifled Discharge Pipe, Dredge *Jadwin*, June 1–6, 1936

Item	Description	Run 4		Run 5	
		Rifled	Plain	Rifled	Plain
1	Effective time, in hours	5.5	5.5	5.0	5.0
2	Average percentage of solids	24.7	18.2	18.1	13.3
3	Average velocity, in feet per second	20.3	21.8	15.8	14.8
	Production, in Cubic Yards per:				
4	Hour	3,808	2,994	2,130	1,464
5	Shaft horsepower hour	2.63	1.97	2.42	1.76
6	Pump speed, in revolutions per minute	175	175	150	150
7	Pump efficiencies (percentages)	68.5	66.8	66.8	67.2
8	Percentage increase in hourly yardage, rifled pipe over plain pipe	27.2	....	45.5	....

Pipe-line velocities were obtained by the electro-saline method with the apparatus arranged as shown in Fig. 13. It will be noted in Fig. 13(*c*) that the terminals were separated by rifled pipe. This fact was responsible for consistent and reasonable results, even when high concentrations of solids were being transported, whereas the method gives highly erroneous results under similar conditions when the terminals are separated by smooth pipe. Data on the distribution of solids across the plain and rifled pipe during these tests have been presented by H. S. Gladfelter,[8] M. Am. Soc. C. E.

Every effort was made throughout the tests to obtain the maximum output of the dredge under all conditions. The results of the tests are shown in Table

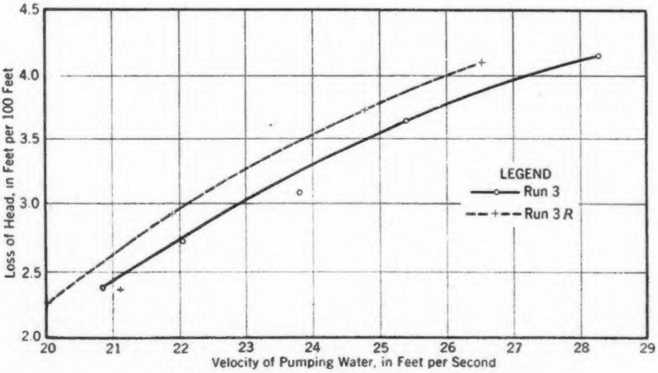


Fig. 14.—Loss of Head in Plain and Rifled Pipe

[8] *Transactions*, Am. Soc. C. E., Vol. 104 (1939), p. 1374.

3, and it will be noted that the greater advantage of rifled pipe over plain pipe is evidenced at the reduced pump speed. This indicated that rifling would be advantageous when used on dredges of low power or to compensate for low pipe-line velocities due to any number of causes. The yardage data (item 4, Table 3) are based on computations made from the percentage-of-solids readings and the discharge velocity.

Fig. 14 shows the comparison of friction losses in plain and rifled pipe lines when pumping water, the loss for the rifled pipe being higher throughout the range. It appears, however, that the situation is reversed when heavy sands or gravel are being transported through the pipes, due to the mixing effect of the rifles.

Although the rifled pipe showed definite advantage during the test described herein, the writer conducted subsequent tests with dredges pumping fine sand and silt in which the plain pipe was equally as efficient as the rifled pipe.

It appears from the foregoing that the conclusions drawn by Mr. Howard from the results of laboratory tests are substantially borne out by tests of dredges made under ordinary operating conditions.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TRANSIENT FLOOD PEAKS

Discussion

By Messrs. Karl J. Bermel, and R. W. Davenport

Karl J. Bermel,[45] Esq. (by letter).[45a]—The rates of runoff that would be necessary to obtain the flow sections from zones of excess runoff are, as the author states (see text supporting Table 2), out of proportion to the flow that could be sustained by the recorded rainfall. The supposition that a maximum momentary flow section greater than could be sustained by the entire rainfall can be attributed for the most part to the formation of flood surges brings to mind other factors that should be considered.

In a problem of this nature the two factors—rainfall, or estimate of rainfall, and runoff, or estimate of runoff—influence the rainfall-runoff relation. A variation in the measurement or estimate of either quantity from that which actually occurred would lead to erroneous results. Since the paper deals primarily with runoff, this factor will be considered first.

Attention should be drawn to the fact that runoff containing debris from a given area cannot be expressed as a percentage of rainfall on that area unless the quantity of transported material is negligible. If it is assumed that the entire rainfall becomes runoff and that there is no transported material in the flow, the runoff would amount to 100% of the rainfall; but if this runoff is increased in volume by transported material, the combined total, as indicated by a flow section, could not be accounted for by rainfall alone. This is particularly true when the runoff from zones of excess runoff contains as much as 50% solids by volume (heading "Debris Content").

From the author's statement, preceding Equation (1), that "No attempt has been made to compute the peak flows" (from zones of excess runoff), it seems apparent that the statements following Table 2—"or 15 times larger than the average flows in the normal zone"; "or 23 times larger than average flows in the normal zone"; and "or 57 times larger than the average flows in the normal zone"—should read "larger than the flow sections in the normal zone."

Note.—This paper by Henry B. Lynch, M. Am. Soc. C. E., was published in November, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: January, 1940, by Ivan E. Houk, M. Am. Soc. C. E.; March, 1940, by Messrs. Gordon R. Williams, and Donald M. Baker; and April, 1940, by Messrs. James M. Fox, F. C. Finkle, A. L. Sonderegger, and Harold C. Troxell and R. Stanley Lord.

[45] California Forest and Range Experiment Station, U. S. Forest Service, Berkeley, Calif.

[45a] Received by the Secretary March 12, 1940.

Reference is made to the statement following Table 2 and the list of average measurements: "A flow of 4,000 cu ft per sec per sq mile requires a runoff at the rate of more than 6 in. in depth each hour, and in these streams would have a section of approximately 130 sq ft for each square mile." Although this statement is true for clear water, it is not a true index of the flows that would be necessary to create their respective flow sections in the zones of excess runoff.

In the foregoing example a value of approximately 31 ft per sec is used as the average velocity; but from the evidence presented in the paper this value is more nearly that of the maximum surface velocity. The average velocity would be less than the surface velocity, especially if the surface velocity is that of a surge, for as Mr. Lynch states (see heading "General"): "With a runoff that is growing greater there is a tendency for the deeper, faster moving water from upstream to overtake the water in front of it."

This is also demonstrated by the following formula[46] in which the speed of the front of the surge is

$$\omega = V + \sqrt{g\left(\frac{A}{B} + \frac{3}{2} Z\right)} \qquad (3)$$

In Equation (3) V = the mean velocity of water in the channel before the surge; A = the original channel cross section; B = the original channel width; g = the acceleration of gravity; and Z = height of the front of the surge above the original water surface. With a reduced average velocity, a flow of 4,000 cu ft per sec per sq mile would require a flow section larger than 130 sq ft per sq mile. Furthermore, when this clear-water flow was bulked by debris, as it would be if it were representative of flows in zones of excess runoff, the flow section would be increased materially, not only by the bulking, but also by a lowered average velocity occasioned by this debris.

As a further consideration of runoff exceeding the flow that could be sustained by the rainfall, variations between measured rainfall and rainfall that is actually intercepted by a watershed may also lead to erroneous rates of runoff expressed as percentage of rainfall.

The major factors that contribute to the error of using rainfall, as measured by a standard U. S. Weather Bureau rain gage, as a measure of watershed interception, excluding the factors of number and location of gages, are the inclination of rain, the gradient of the watershed slopes, and the aspect of these slopes with respect to the rain inclination. The following theoretical considerations are presented to illustrate the fallacy of using rainfall, as recorded by a standard rain gage, as a criterion of rainfall intercepted by a watershed.

In Figs. 19 and 20 the following quantities are used: Area of topography, 1 sq ft (horizontal projection); area of rain gage, 1 sq ft; rainfall, 1 in. per hr for a duration of 1 hr, represented by a uniform spacing of 10 lines per ft on a line at right angles to the direction of rainfall; and inclination of rain and slope of topography are assumed as 45° for purposes of illustration.

In Fig. 19(*a*) the topography is at any gradient and the rain falls perpendicularly. In Fig. 20 the topography is level and the rain falls at any inclination. These are the only conditions under which recorded rainfall is

[46] "Hydraulic Structures," by Armin Schoklitsch, Vol. I, p. 123; translated by Samuel Shulits, A.S.M.E., 1937.

a measure of surface interception. It is apparent in Fig. 19(*a*) that the rainfall intercepted by either the level or the sloping topography or the rain gage is the same—that is, a depth of 1 in., or, on the basis of a horizontal area of 1 sq ft, 144 cu in. The depth on the slope area is obtained as follows: $\frac{b}{c} = \cos \alpha$; and $c = \frac{b}{\cos \alpha} = \frac{1}{0.7071} = 1.414$ ft. The slope area equals $1.414 \times 144 = 203.62$ sq in.; the ratio $\frac{\text{Volume of rain}}{\text{Slope area}} = \text{depth per area in inches} = \frac{144}{203.62} = 0.7071$ in.

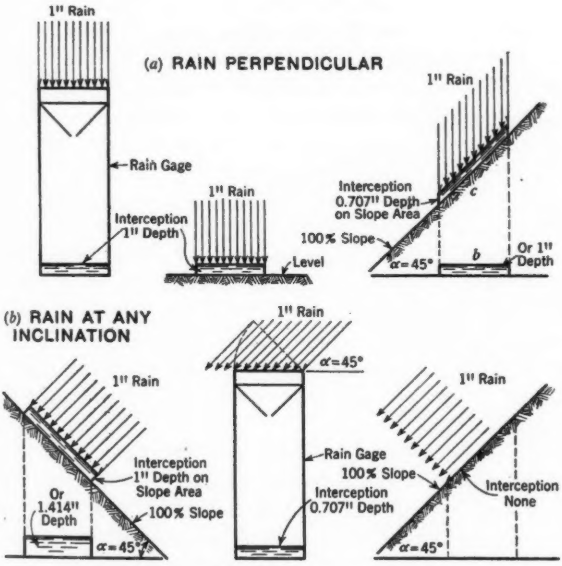


FIG. 19.—TOPOGRAPHY AT ANY SLOPE

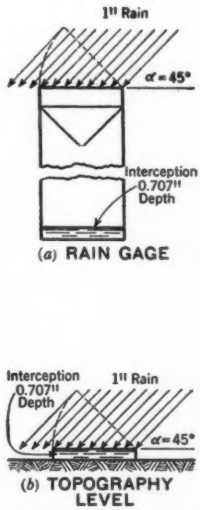


FIG. 20.—TOPOGRAPHY LEVEL; RAIN AT ANY INCLINATION

In Fig. 19(*b*) it is apparent that the rain gage and the level topography intercept equal quantities of rainfall, not taking into consideration differences due to such factors as wind currents and splash. The rainfall that is intercepted, however, depends upon the area capable of interception, which is the horizontal area projected at right angles to the direction of rainfall. The projection of any plane area upon another plane is the product of that area and the cosine of the angles between the planes. Since the horizontal area is 1 sq ft and the cosine of 45° = 0.7071, the projected area equals 1 times 0.7071 = 0.7071 sq ft or 0.7071 times 144 = 101.82 sq in. Since 1-in. depth of rainfall is intercepted on this projected area, the volume of rain intercepted equals 101.82 times 1 = 101.82 cu in.

Fig. 19(*b*), in which the topography is at any gradient (except level) and the rain falls at any inclination, illustrates the error that can be introduced by the use of recorded rainfall as a measure of surface interception. Where

the rain falls parallel to the surface of the topography (Fig. 19(*b*), right) no interception can occur; the rain gage, however, records 0.7071-in. depth or 101.82 cu in. of rain (see discussion of Fig. 20). Where the rain falls perpendicularly to the topography (Fig. 19(*b*), left), the slope area (203.62 sq in.—see discussion of Fig. 19(*a*)) is capable of intercepting 1 in. of rain, or 203.62 cu in., or, on the basis of the equivalent horizontal area, 1.414-in. depth, or 203.62 cu in. Again, the rain gage records 0.7071-in. depth, or 101.82 cu in. There is a difference of 101.60 cu in. of rain between rainfall intercepted by topography and rainfall recorded by the rain gage, an approximate difference of 100%. Where inclination and watershed gradient are greater than in the cases illustrated, the differences may be greater; and, likewise, where the rain inclination and the watershed gradient are less the differences are less.

These instances are not intended merely to indicate the degree of influence of rain inclination; they reveal the errors that can occur in the present method of using standard rain-gage readings as an indication of watershed interception.

On a watershed these extreme conditions of difference are compensated by opposing slopes, but where the aspect of the watershed as a whole faces the rain front and the rain falls at an inclination, it is conceivable that the watershed intercepts more rain than the amount indicated by a standard rain gage and that runoff rates would exceed this recorded rainfall. In most cases the drainage areas shown in Fig. 2 and listed in Table 1 as "zones of excess runoff" face the general direction of the rain front, and they tend to intercept more rain than the quantity recorded. On the other hand, the aspect and lower gradient of Sycamore Canyon, described under "Steepness of Canyons," might have been favorable to actual interception nearly equal to observed rainfall and consequently to more normal runoff.

In predicting runoff rates, or in assigning various coefficients to runoff, it is necessary first to be able to ascertain the amount of rainfall intercepted by a watershed. Rainfall records at best provide only an indication of the actual rainfall for an area. However, rainfall recorded by a slope rain gage would be a better indicator of actual watershed interception, especially in steep watersheds. For the slope gage to intercept rainfall representative of the area it samples, the plane of each gage must be placed in the average plane of the topography it is to represent. The factors of slope, aspect, and rain inclination need not then be considered. However, the number of gages, the spacing, and the other considerations that arise in initiating installations for proper sampling of an area will present even more difficulties than are now encountered in the installations of standard rain gages.

R. W. Davenport,[47] M. Am. Soc. C. E. (by letter).[47a]—The engineers of the U. S. Geological Survey, who are engaged in determining the flow of surface streams, and in correlating stream flow with rainfall and other influential factors, will find this paper especially interesting.

The determination of the flow of surface streams relates, of course, to the entire range of flow, from low water to floods. The objectives of the U. S. Geo-

[47] Hydr. Engr. in Charge, Div. of Water Utilization, U. S. Geological Survey, Washington, D. C.

[47a] Received by the Secretary March 18, 1940.

logical Survey are to collect and publish, in such detail as is practicable, the information that will be needed by users of the records. An increased interest in flood data, coincident with a period of greater prevalence of outstanding floods, has been met by a pronounced effort to satisfy the needs of all concerned as adequately as possible. Consequently, greater emphasis is being given to supplying flood data in the annual reports on surface-water supply, and several special flood reports and compilations have been published. Valuable contributions to special reports of that kind have been made by many public and private agencies.

The general objective of the special reports is to furnish, so far as practicable, records of stage and discharge at regular gaging stations throughout floods of major consequence at sufficiently close intervals to permit a satisfactory reproduction of the actual hydrograph, and to furnish peak discharges of the flood at miscellaneous places of interest where continuous observations are not available. Determination of dependable peak discharges is obviously a problem of major importance.

Generally, in all except the largest rivers, flood peaks have the characteristic of being comparatively brief and transitory and of occurring in many streams almost simultaneously. Because of the usual interference with, or paralysis of, transportation facilities by abnormally high floods, the widely separated locations of gaging stations, and limited engineering personnel, it is rarely possible, particularly on the smaller streams, to have an engineer on the ground to observe the passing of the peak. Because of the wild and destructive behavior of floods, as well as the great quantity of debris often carried along by the water, it is commonly quite impossible to obtain a direct measurement of peak discharge. This situation requires that efforts be made to determine peak discharge by obtaining as much physical evidence reflecting the magnitude of flow as possible for analysis by suitable hydraulic formulas, such as the Manning formula (using slope and cross-sectional area), the formula for flow over weirs, the formula for flow through contracted openings, and others of less general utility. The factors involved are too often far outside the range of guiding experience with the formula used. In most instances the exercise of a large degree of mature judgment, which can be gained only by wide experience, is required.

The complex problems related to determination of peak discharges is particularly challenging in floods of the so-called "cloudburst" type. Such floods occur with varying degrees of frequency in different sections of the United States. They are associated mostly with comparatively small drainage basins. Where they have occurred on streams draining larger basins, it appears that the excessive rainfall which caused them was largely limited to relatively small parts of the basins. Cloudburst floods in the arid regions are commonly associated with the movement of large quantities of debris of various kinds. In humid regions the movement of debris, although characteristic, tends to be a less prominent feature.

If a surge-like flood of water, unmixed with debris, were passing down a channel that previously was essentially empty, the magnitude of the surge would tend to diminish progressively because of steady depletion by channel

storage. The surge could be maintained at its original magnitude, other conditions remaining favorable, only if tributary inflow were properly timed and equivalent to the depletion in channel storage. Thus the conditions requisite to the persistence of a surge of water free of debris in a natural channel seemingly include a peculiar synchronization or combination of high rates of inflow from tributary drainage. A similar coincidence of high inflow is even more essential to the original generation of a surge.

Studies of the relation of rainfall and runoff have indicated that appreciable quantities of rain are accounted for as surface detention—water which wets the ground surface and fills the infinite number of more or less minute depressions, as distinguished from infiltration, or absorption, and runoff. If a torrential rainfall should happen to come at a time when surface detention was relatively large, due to favorable antecedent conditions such as the accumulation of vegetation and litter, it is conceivable that a substantial part of such surface detention might quite abruptly become available as runoff through breakdown or failure of the obstructions storing the water. The runoff from the torrential rainfall would therefore be augmented perhaps by a substantial percentage through the sudden release of surface detention, or the failure of innumerable miniature reservoirs. The drainage area over which such phenomena would be likely to occur simultaneously would seemingly be relatively small. The writer conceives that an event of this kind may be one of the factors in generating surges in cloudburst floods.

If a flood of water unmixed with debris were collecting in a channel system and passing down the main channel as rapidly increasing discharge, and if in the forepart of the flood a large quantity of mud, gravel, boulders, and all kinds of trash were introduced, the effect on the flow would be pronounced. With debris composed of finer grades of mud and gravel the moving mass would in some measure be comparable to a liquid, but a liquid that behaves quite differently from clear water. With coarser debris consisting of boulders, logs, and trash the upset in hydraulic behavior would also be striking. The velocity would be slowed down by absorption of energy needed to carry the debris down slopes flatter than the dry angle of repose or by reduced liquidity or greater viscosity and consequent internal friction. It would tend to create a relatively slow moving mass which would be overtaken and enlarged by the more rapidly moving water and debris that is following. In a sense it would behave like a dam.

Obviously, the effect of any other factors tending to produce a surge of flow in the channel, such as torrential rainfall, favorable concentration of runoff, and other influences, would be increased by a substantial increment of debris. Probably this kind of phenomenon is largely responsible for the frequently reported wall-like aspect of the front of the wave and other surge-like features common to cloudburst floods.

The actual discharge represented by such a moving dam with its following pond of water and debris would seem to depend not only on the rate of movement of the dam and the rate of flow of the oncoming water, but also upon what portion of such oncoming water goes into temporary channel storage behind the moving dam. Rupture of the dam and abrupt release of the retarded water

would doubtless result in very high rates of discharge until impeded again by the accumulation of more debris and further reduction by channel storage. The conditions which arise from the breakdown of one surge of this kind may react to form another. The widening or narrowing of a channel may also have an important influence on the manner of progress of a surge. Unfortunately, the lack of even the most fragmentary field data on these phenomena prohibits the crudest form of analysis. The author's finding that floods of the "cloudburst" type yield momentary runoff peaks entirely out of proportion to the rates of rainfall is apparently based on information other than the direct comparison of observed rates of rainfall and runoff. In surge-flow peaks the effects of channel phenomena may obscure any satisfactory quantitative relation with rainfall rates.

The cross-sectional areas listed in Table 1 may be less indicative of flowing water than of a more viscous, slower flowing liquid heavily loaded with debris. It is obvious that the movement of large quantities of debris may seriously complicate the determination of rates of flow by hydraulic formulas or any other means.

The following quotation from a report[48] of the U. S. Geological Survey on the New York State flood of July, 1935, discusses some of the difficulties encountered in making flood determinations, and issues cautions as to their use:

> "The application of formulas and coefficients used in the computation of the flood flows was made with a full appreciation of the limitations of scientific knowledge of the behavior of streams under unusually extreme conditions such as those of the July flood, and is believed to be consistent with good engineering judgment. Many of the streams undoubtedly carried enormous quantities of debris. The effect of this debris upon the applicability of the laws and formulas generally accepted as governing the flow of water is problematic. The same statement applies to the effects of sediment, entrained air, turbulence, excessive slopes and velocities, and other factors, which occurred in a degree far outside the field of ordinary experiment and experience. * * * It is believed that, through the reaches selected for the determination of flood discharge, the debris moved downstream in such a manner as to cause very little if any reduction in apparent area or water capacity of the channels. For the purposes of this report it has been assumed that the water surface of the streams was represented by the high-water marks indicated on the banks, that the channels as surveyed had remained substantially unchanged throughout the flood, and that the flow conformed to the laws of the flow of water expressed by the formulas selected for the determination of the particular flood discharge. The results thus obtained are believed to be in the most useful form and of such value in the planning of flood-protection measures as to warrant their publication as the most reliable data that can be supplied. However, any user of the data is cautioned to keep in mind the method of derivation and to make such allowance therefor as may seem appropriate."

With all the attendant difficulties and resulting uncertainties that may exist in the determination of discharge by direct measurement, or indirect methods, it should be recognized that certain inaccuracies are inherent and recognizable in the determination of supply by rainfall observations. Unfor-

[48] "The New York State Flood of July, 1935," by Hollister Johnson, Assoc. M. Am. Soc. C. E., U. S. Geological Survey, *Water Supply Paper No. 773-E*, p. 251.

tunately, the measurement of amounts, duration, and intensity of precipitation is fraught with some difficulty, particularly on an areal basis such as that usually associated with cloudburst floods.

Approaches to the problem on the broad basis of available knowledge of rainfall rates, infiltration rates, etc., may contribute materially to an understanding of the flow characteristics of transient flood peaks. Investigations of the volume of water required to transport measurable deposits of debris may be illuminating.

The author has presented interesting and valuable information concerning floods of the cloudburst type. From the standpoint of a satisfactory and complete understanding of the associated phenomena, the exploration of the field has only begun. With the accumulation of observations and experience, both in the field under natural conditions and in the laboratory, the prospects for better understanding are promising.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

CHANNELIZATION OF MOTOR TRAFFIC

Discussion

BY MESSRS. JULIAN MONTGOMERY, R. M. REINDOLLAR, IRVING MACK, BRUCE D. GREENSHIELDS, T. W. FORBES, AND JAMES S. BIXBY

JULIAN MONTGOMERY,[24] M. AM. SOC. C. E. (by letter).[24a]—The study and analysis of traffic flow are requisites of highway design; and although the operation of a highway system is gaining in momentum, it is not a recently created feature of motor transportation. Mr. Kelcey emphasizes this fact in his references to the research of William Phelps Eno[3] and Fritz Malcher.[6]

As early as 1911, in England,[25] Charles Mulford Robinson declared that "increase in complexity, irregularity, rapidity, and danger of the traffic stream have made transportation needs no longer merely matters of widening streets and untangling old networks to provide for volume." This is an interesting observation in view of the date of publication. It shows the definite thought directed toward traffic flow in the early part of this century. This trend in channelization is even more confining than Mr. Kelcey's reference to caravan routes, etc.

The word "channel" has been associated primarily with water transportation. By adding "ization," the word has come to be allied with motor transportation. It appears, therefore, that Mr. Kelcey's definition should be expanded to read, "Channelization is an orderly confinement of movement of vehicles to reduce the number of reference points made by overlapping wheel-mark patterns." An addendum to this renovated definition might include reference to friction areas. That reference marks should be kept to a minimum is well known; that the friction area increases as the angle of crossing decreases

NOTE.—This paper by Guy Kelcey, M. Am. Soc. C. E., was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1940, by Messrs. W. L. Waters, C. J. Tilden, and T. M. Matson; and April, 1940, by W. W. Crosby, M. Am. Soc. C. E.

[24] State Highway Engr., State Highway Dept., Austin, Tex.

[24a] Received by the Secretary March 14, 1940.

[3] "Street Traffic Regulation," 1909, p. 7, Sections 437 and 440, pp. 10, 11, 28, 29, and "The Science of Highway Traffic Regulation," 1920, pp. 26–39, Eno Foundation for Highway Traffic Control, Saugatuck, Conn.

[6] "The Steady-Flow Traffic System," by Fritz Malcher, Harvard City Planning Studies, Vol. IX, 1935.

[25] "Width and Arrangement of Streets," Chapter 5, by Charles Mulford Robinson, Engineering News Publishing Co., London, England, 1911.

is also well known. For example, the overlapping space of patterns made by vehicles crossing at right angles is less than the area made by vehicles crossing at 30° and 15° angles.

Mr. Kelcey neglects to inform his readers that the position of the medial strip (as in Fig. 19) governs the turning radius of vehicles with subsequent control of the speed of the vehicle. Medial strips flush with the curb line of the intersecting street provide short radii and slow speeds; medial strips not extending to curb lines increase such radii and turning speeds.

In commenting on the inefficiency of an intersection due to lagging of motor vehicles during interval changes of stop-and-go traffic signal lights, Mr. Kelcey has encouraged the writer in his belief that efficiency of design is measured by highway users in terms of mobility. Design failures—accidents—are not of so much concern to the majority of motor vehicle operators as continual movement. A steady flow, as illustrated in Figs. 47, 48, and 49, of necessity retards the speeds of vehicles, but it does not demand a full stop. In man's desire for mobility, he measures road benefits in continual movement, even if such movements may mean speeds of 10 and 15 miles per hr.

Mr. Kelcey's reference to driving in advance of one's vehicle as "heads up" is what is termed "Road Focus." [26] At speeds of approximately 30 miles per hr a driver's eye is focused only a few feet in advance of his car, but his range of vision extends horizontally several feet beyond the limits of a normal right of way. At high speeds of 60 to 70 miles per hr, the driver's eye is focused far in advance of his car and his range of vision is concentrated on a limited area. Today the demand is for speed. Subsequently, as the engineer designs for speed, he also designs for the visual characteristics of the motor vehicle operator. It appears, therefore, that the engineer carries the responsibility of providing channels for a correct route. This correlates with Mr. Kelcey's suggestion that "drivers will follow the lines of least resistance and use the correct one." If designers are to be guided by such a policy (and the writer thinks that they should), then the modern grade separation commonly called "clover-leaf" is altogether too complicated for the average motor vehicle operator; and although some advocates believe a simpler design should be substituted, it appears that a clover-leaf is the only adaptation providing elimination of both medial and intersectional friction. It seems the driver confusion of this design could be reduced materially by a standard for all highway departments. In providing only the correct route, channelization, when fully adopted, may eliminate both traffic signal lights and warning signs, leaving only directional signs or colored sections of the pavement (Fig. 49).

Mr. Kelcey cites several cases of the failure of laws in adjusting the needs of the motorists. The physical channelization of motor vehicle flow will shift the responsibility of the operation of the highway system solely to highway engineers. This is encouraging because of its practical application as compared to legislative regulations. In Texas, engineers are anxious to develop a speed zone program for obsolete highways; however, they are restricted because of the state speed limit of 45 miles per hr which prohibits posting speeds

[26] "American Highways and Roadsides," by Jac L. Gubbels, Houghton Mifflin Co., publishers, Boston, pp. 22–23.

in excess of this limit. On the other hand, speed studies conducted by the Texas Highway Department[27] reveal that 39% of the operators of passenger cars exceed that speed limit. Speed zoning is handicapped by antiquated laws. Figs. 12 and 13 substantiate the confusion between law and practice. Fig. 12 represents a Texas state law whereas Fig. 13 represents the common practice of most Texas motor vehicle operators. This practice is more advantageous in expediting the safe movement of motor vehicle traffic, as the number of reference points, or crossings, is reduced from eight to four.

The form of rotary traffic,[6,28] graphically expressed in Fig. 48, is a feature of design which engineers are gradually adding to divided highways in Texas. It is their thought that openings through the medial strip should be governed by the designed speed of the principal, or divided route, in such a manner that traffic may reverse its direction at openings spaced in accordance with accelerating and decelerating distances.

Under "Driving Factors: Merging, Intersecting, and Opposing Movements" Mr. Kelcey declares, "A traffic circle (Fig. 5) with a diameter, of several hundred feet, would provide a merging movement of vehicles which would be essentially parallel." First consideration of a rotary traffic circle[29] should be given the merging distance between the egress and ingress legs of the radial roads. This distance is a segment of the circle. It is the writer's belief that by providing sufficient merging distance between radial points, the designer has assured a factor of safety that cannot be ascertained from any other sole feature of design. The length of the segment of the circle depends upon an accelerating distance which would enable the overtaking and passing of vehicles on the arc of the circle between radial points. Such a design approach does not indicate that an increased diameter would provide a merging movement. For example, at the intersection of five roads, the angle of the intersecting roads might warrant the circle to be offset from the center of the intersection in order to provide minimum merging distance for one of the segments of the circle. By such a system, the radius might be kept down to a practical length and still provide the required merging distance. It appears that there is a limit to the radius of a traffic circle. A limit could be established for rotary-circle radii by changing the alinement of one or more of the radial roads in order to provide required merging distance. This could be accomplished according to the principles of design currently used in the decelerating of motor vehicles approaching traffic circles. In investigations of traffic circles in Texas, engineers have found that safe merging distances are more important than any other single design consideration.

In 1939, 22% of the motor fatalities in Texas were classified under medial friction whereas approximately 8% resulted from intersectional friction. Definite channelization might have reduced these percentages substantially.

---

[27] Texas Highway Dept. Information Exchange, Issue 69, May 15, 1939.

[28] "Unfit for Modern Motor Traffic," *Fortune*, August, 1936, p. 85–92, 94, 96, and 99.

[29] "Manual of Instructions, Traffic," Texas Highway Dept. (in preparation).

R. M. Reindollar,[30] M. Am. Soc. C. E. (by letter).[30a]—The broad scope of Mr. Kelcey's paper, the capable and comprehensive manner in which he makes his presentation on channelization, and the exhaustive information contained therein disclose the time and study which he has devoted to this subject. The control of traffic movement over the highways, involving the direct action of so many individuals in all walks of life, is certain to provoke considerable controversy.

In his calculations of loss of time for traffic passing through intersections controlled by traffic lights, Mr. Kelcey does not take into consideration the compensating factor which, by reason of the fact that cars are "bunched up," allows them to pass through an intersection with closer intervening space between vehicles. At lower speeds, this factor permits a greater number of vehicles to pass a given point within a specified length of time.

The author's description of eddy currents caused by unexpected and irregular movements of reckless drivers is excellent, and should be read by all investigators, as well as trial magistrates, passing on collisions. In many cases one motorist is the innocent victim of actions of another, and not only sustains damage to his vehicle, but is taken to the traffic court and prosecuted for reckless driving, whereas the motorist actually responsible for such a collision is rarely apprehended because of the fact that he is not physically involved.

The author's suggestion to simplify the problem of crossing an intersection by changing the right-of-way law—giving the driver to the left the right of way at intersections, instead of the driver approaching from the right—is a subject on which there will be considerable disagreement. Although the facts in favor of such a change in regulations are ably presented, consideration is not given to the resulting delays to main-line, left-turning traffic that would be caused by ceding the right of way to a motorist approaching from a side road on the left. In addition, it is possible, with four cars from all approaches arriving simultaneously, to have a complete tie-up as described by the author for the right-hand right of way. It is a subject, however, which merits additional consideration and study.

The traffic flow as shown in Figs. 19, 21, 23, and 26 is materially improved by the channelization shown, but more complete control would be obtained by the installation of islands at all approaches, as there is still a possibility of short cutting across opposing traffic lanes on two legs of the intersection. In Figs. 24, 31, and 32, where islands are established for a predominant left-turn movement, they should be wide enough to permit car storage, if right-of-way costs are not prohibitive; and decelerating lanes should be provided so as not to impede through traffic on the main road.

With reference to the elimination of cross traffic at street intersections on divided highways, which are separated by a narrow center island (see Fig. 48), the advantages of the merging flow might be completely offset by the resulting confusion at the point of left or U-turn between intersections, as decelerating lanes and adequate storage space are not provided. The author describes this treatment in his arrangement as shown in Fig. 49, but the contention is that,

[30] Asst. Chf. Engr., State Roads Comm., Baltimore, Md.

[30a] Received by the Secretary March 11, 1940.

unless the features incorporated in this latter development can be applied, crossings at street intersections should not be eliminated.

In conclusion, it should be stated that no highway engineer can read Mr. Kelcey's paper without immediately visualizing many intersections, with which he is familiar, where, by the application of some of the principles as set forth, certain hazards can be eliminated, and a freer movement of traffic can be promoted. It should be read not only from the viewpoint of the economies involved, by reason of savings in time and operating costs through the application of the principles so ably presented, but also for its value as to safety features, which make for reduction in accidents, with their resulting toll in injuries and fatalities.

IRVING MACK,[31] ESQ. (by letter).[31a]—A number of suggestions were not included in the paper which are often helpful to officials who must do the best they can with existing roadways and who, although agreeing with the principles of channelization, find the problem of actual application a little difficult. These matters have to do with the approach to such problems and how the use of stop-and-go signals may be related and applied, where necessary, to a channelization treatment.

The Approach.—Surprizes, ambushes, deceptions, and illusions must be eliminated in the design of any traffic facility so that there will be no unexpected hazard, uncertainty, hesitancy, or confusion under any circumstances of use. Unless it is impossible to avoid, drivers must not be confronted with the choice of more than one of two courses at one time.

In studying the problem of existing pavement area, it is important to lay it out to scale. It is difficult to visualize correctly (as many fail to do), on the ground, the proportions of all aspects of a traffic area or other situation which is more complicated than a simple intersection.

The traffic engineer should study irregular traffic movements in an existing pavement area by observing marks left by the tires of vehicles. If tire marks are not clear, he should throw some sand about the intersection, or dry cement in approach lanes, and let the traffic of the next half hour write its own story.

If the engineer is in doubt as to the correct treatment, tests may be made by one of several methods. In snow country areas the exact shape of islands called for in a design may be filled in with heaps of snow by snowplow crews. A car, motorcycle, barrels, or a heap of sand may be used to test out a proposed treatment. Markers of iron or concrete also may be used for experimental observations, in various positions, for a few hours or days.

A channelizing treatment that has been properly designed and installed invariably exceeds its designer's expectations by working much better than was hoped for. If any confusion develops, at first, it will shortly disappear after local drivers, who have formed fixed habits with respect to the location, become accustomed to it. If it is a good treatment, strangers will know what to do at once.

---

[31] Mgr., Electric Signal Div., Signal Service Corp., Elizabeth, N. J.

[31a] Received by the Secretary March 18, 1940.

The matter of public relations should not be overlooked. The engineer should see to it that local drivers know why, when, and how; and afterward he should tell them the results, too.

Stop-and-Go Signals.—The stop-and-go signal is an excellent device when properly used. In fact, its proper use should be greatly extended. Unfortunately, it has often been misapplied. Channelization helps to eliminate the abuses and to assist stop-and-go signals in the performance of their proper functions.

Since channelization permits continuous, safe flow with minimum stoppage, congested conditions, which are often more an evidence of paralysis than of overload, are usually relieved by its application. It is frequently possible, therefore, to correct irregular traffic movements at troublesome intersections to such an extent by channelization that stop-and-go signals, if needed, may be operated as such only during heavy load periods and turned to flashing by time clock, or reverted to manual operation at other times.

When stop-and-go signals are used in connection with a channelization treatment on islands, they signalize the islands whether operating as stop-and-go or flashing. They are also in the most advantageous position to be seen by pedestrians and to control traffic. Traffic control and regulation, pedestrian refuges, and other values are thus all provided at minimum cost by the one treatment.

It is unfortunate, perhaps, that, in his conclusion, Mr. Kelcey stresses so greatly the use of the principles of channelization in new design and neglects to emphasize the advantages of continuous flow and safety to be derived from the application of these same principles to existing roadways.

In the writer's experience, the principles of channelization may be applied to many "Eighteenth and Nineteenth Century" streets and roadways at slight expense and with surprizing results. In fact, such application is often so simple and inexpensive that successful treatments of older roadways frequently await only the effort of studying out the solutions to substitute for or to postpone costly reconstruction.

BRUCE D. GREENSHIELDS,[32] ASSOC. M. AM. SOC. C. E. (by letter).[32a]—The very excellence of Mr. Kelcey's paper serves to emphasize the need for accurate design data obtained by careful field studies. It is generally accepted by highway engineers that channelization does increase "the utility and safety of automobile transport by simplifying traffic flow and by eliminating confusions, conflicts, and eddy currents," but there is no agreement as to whether the turning radius at an intersection should be 20, 30, or 40 ft, or some other distance. Vehicle speed, of course, is definitely limited by the turning radii.

The minimum turning radius of cars, trucks, and buses varies from approximately 30 to 50 ft. Joseph Barnett,[33] M. Am. Soc. C. E., gives 28 ft as the minimum turning radius for cars, and 45 ft for buses and trucks. On a parkway limited to passenger cars it may be expected that the turning radii of channels will be less than where truck and bus traffic is permitted; but sharper

[32] Associate Prof., Civ. Eng., School of Technology, Coll. of the City of New York, New York, N. Y.

[32a] Received by the Secretary March 22, 1940.

[33] "Highway Intersections," by Joseph Barnett, *American Highways*, January, 1940, p. 18.

curves reduce speed and thus limit highway capacity. Furthermore, it must be kept in mind that it is impossible to drive a vehicle in a path composed of simple curves and straight lines. Always the car is eased into a curve by a gradual turning of the steering wheel. The higher the speed, the more gradual is the turning. Too abrupt a turn would invite disaster. Thus, the higher the speed, the longer must be the "easement curve."

The most advantageous path can be ascertained best by field observations. The camera offers a very good means of making these observations. In interpreting the field data it is important to remember that it is not the mean or average driver's performance that is most important but rather that of the 80 or 85 percentile driver.

The physical factors involved are susceptible of analytical analysis but the human factors are not unless statistical analysis is included. Whether the true spiral, the three-centered compound curve, or, in certain cases, the parabolic curve is most suitable will depend upon the field observations.

The channel of the highway, in contrast to the railroad, must be followed by the willful direction of the driver. This means that the channels must be clearly visible. Light reflecting surfaces and adequate lighting at congested points are necessities for safety.

Unless the dividing islands are to be used for pedestrian safeguards, the writer believes that stop-and-go signals should not be installed on pedestals on these islands. Almost every day somewhere in New York City the base of a signal or light pole is struck and broken by a motor vehicle. In so far as it is possible, the islands should be so designed that cars striking them will not be injured.

T. W. Forbes,[34] Esq. (by letter).[34a]—Compliments are due the author, not only for systematizing the wide variety of physical layouts which are met in practical channelization work, but also for introducing a discussion of the very important human factors involved. His conclusion that plans for future designs must take into account driver characteristics and needs are points very well taken. Many engineers have recognized the importance of human factors, but it is of especial significance to see it included in such a paper. In fact, the points mentioned in the paper are worthy of amplification in the following respects:

1. On careful consideration, it appears that the main and outstanding difference between structural highway design and traffic design is that the latter must take account of the human factor.

This difference has been recognized by leading design engineers.[35, 36] Structural design must deal with materials, vehicle weights, and stresses, but need not consider how drivers will use the highway or structure. Engineers working in the traffic field, however, have been forced to recognize the variables intro-

[34] Research Asst. (Asst. Prof.), Bureau for Street Traffic Research, Yale Univ., New Haven, Conn.

[34a] Received by the Secretary April 3, 1940.

[35] "The Modern Express Highway," by Charles M. Noble, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 1068; "Thoughts on Highway Design Research as Related to Safety of Vehicle Operation," by Charles M. Noble, *Proceedings*, Highway Research Board, 17, 1937, p. 242.

[36] "Needed Research for the Determination of Sight Distance," by Joseph Barnett, M. Am. Soc. C. E., *Proceedings*, Highway Research Board, 17, 1937, p. 111.

duced by the human factor. Unfortunately, the high variability and difficulty of controlling these factors have led to the general feeling that the human factor is nebulous and impossible to handle.

2. If properly approached, however, the human variables can be handled on a definite and practical basis.

It is true that the human reactions that occur in the driving habits of the general public are varied. However, they are subject to certain general laws, and to definite limitations. By combining good physical or engineering research technique with a knowledge of physiology, psychology, and optics, it is possible to determine certain minimum values which will be of practical use in traffic design. The variability of the subject matter must be taken into account by obtaining a sufficiently large number of cases, and a sample representing the driving of the general public. As examples of practical research methods on human factors in traffic design, two of the writer's studies may be cited.[37, 38]

In his paper, the author mentions the general laws of habit, of "least energy," the necessity of pitching instructions for the fairly low average intelligence level, and the fact that driving is directed some distance ahead. These are all good and very pertinent, but can be even more specific.

3. It is not overexaggeration to say that the human factor is the cause of the problem with which this paper is dealing.

The author recognizes this fact in his use of the following expressions throughout the paper: "Reducing uncertainty," "reducing shuttling," "reducing the number of alternate paths," "the necessity of looking two ways," and other similar expressions. To be more specific the writer proposes to examine the fundamental reasons giving rise to this uncertainty and shuttling by the motorist.

In order to analyze the cause of the difficulty, the traffic engineer must understand the kind of mechanism with which he is dealing.

First, the mechanism (the driver) has an angular field of vision of approximately 180°, but his clear vision is limited to a cone of about 5° in the center of this field. He throws this around like a searchlight, and his clear visual attention at any instant is limited to this relatively small field. The driver, then, should be considered as if he were operating his car in the dark by the aid of a searchlight with a bright center beam of 5°, and a very diffuse and low-intensity halo filling the remainder of the 180°.

Second, the "mechanism" requires a certain amount of time to operate. This time period varies in length with the number and complexity of the operations which are involved in the same fashion that the switching time for a call going into an automatic telephone exchange depends on the number of stepping relays to be actuated and the number of exchanges through which the call must go before it arrives at its destination. The simplest operation is "a simple perception-and-reaction time" which involves an ingoing impulse,

[37] "Overtaking and Passing Requirements as Determined from a Moving Vehicle," by T. M. Matson and T. W. Forbes, *Proceedings*, Highway Research Board, 18, 1938, p. 100.

[38] "Methods of Measuring Judgment-and-Perception Time in Passing on the Highway," by T. W. Forbes, *loc. cit.*, 19, 1939, in press.

a simple central switching operation, and an outgoing impulse. Simple brake-reaction time averages from 0.4 to 0.5 sec. This involves only one reaction, so that no elaborate central station switching is required. However, if the driver must choose between several signals and several courses of action "choice perception-and-reaction time" is involved, in which more complex central switching is required. This is illustrated when the driver sees a red signal ahead, perhaps on a rear car stoplight, and must decide whether to step on his brake or to turn his steering wheel and pass the car ahead. Such choice-reaction times, in fairly simple situations, have been demonstrated to be from 0.75 to 1.0 sec in extent. Under complex situations, they are probably greater than this.

The most complex central station switching, calling into play several different exchanges, occurs when the driving situation calls for a definite skilled judgment on the part of the motorist. This type of reaction is illustrated when the driver must judge the speed of oncoming cars, the speed of his own car, and the speed of the car ahead of him, and decide whether or not it is safe to pass. The time involved in such complex mental "switching operations" or "perception-and-judgment time" is from 2.8 to 3.5 sec.[38]

Now, apply these considerations to the ordinary open right-angle intersection, assuming two 40-ft streets, and a speed of 30 miles per hr. To be conservative, use the one-second "choice-reaction time." This will require approximately 45 ft of travel for a car before the driver will be able even to put his foot on the brake, or start to turn his steering wheel. Therefore, if some action by another car occurs just as he is entering the intersection, he must, of necessity, be across the intersection before he can start to change the course or the velocity of his vehicle—that is, unless he happens to be looking at the vehicle, and the emergency is so obvious as to reduce his reaction to a simple and automatic one, in which case he will be halfway across the intersection.

To make matters worse, examine the 5° cone of searchlight beam, representing his clear visual field. It has been shown that it takes from 0.6 to 0.8 sec to flick this visual searchlight from one point to another and back again.[39] Actual seeing takes place only during the time that the searchlight is resting "fixated" and not moving. The smallness of the cone indicates that in a right-angle intersection each of the three entering ways must be fixated separately when the driver is just entering his own street. Thus, he must give his attention to at least three points at a cost of at least 0.6 sec each; and, if anything unusual happens, his own central switching process may take from about 1.0 sec to as many as 3.0 or 3.5 sec.

When it is considered that each intersection represents not only three entering ways, but two entering lanes of traffic on each, and at least three different general paths which each stream of traffic might take (straight, left turn or right turn), it is apparent immediately that even an ordinary right-angle intersection of two 40-ft streets, with a continuous flow of traffic at about 30 miles per hr, presents a task for the human switching mechanism in which split-second reaction is impossible.

[39] "A Method for Analysis of the Effectiveness of Highway Signs," by T. W. Forbes, *Journal of Applied Psychology*, 23, 1939, p. 669.

This is the problem which, as the paper so ably points out, is often solvable by channelization. Fundamentally, then, how does channelization solve the problem? First, it localizes conflicts to certain points, small enough to be taken in by the 5° cone of clear vision. Second, it spaces these points so that the human switching mechanism can deal with one of them at a time. Third, it reduces the time necessary for the mental switching process at each point of conflict by reducing the number of choices necessary and by eliminating or simplifying the judgment that must be made. In other words, instead of having to deal with "perception-and-judgment time" the engineer can deal with "choice-perception-and-reaction time," that is, he can reduce the time magnitude from 3.0 sec or more to about 1.0 sec, plus.

Sometimes it has been found in the field that certain channelization has not been effective. If this unsuccessful channelization is submitted to analysis on the foregoing basis, it will be found that channelization succeeds in proportion as it accomplishes the foregoing three objectives. Thus, a definite cause of the problem has been indicated and a definite basis outlined on which the successful solutions may be analyzed in physical terms, but in the light of the knowledge of the "human mechanism."

> 4. However, the most carefully designed channelization or other traffic design may be made ineffective if certain other human factors are not taken into account by the engineer.

The most important of these is what may be called "mental set or expectation" of the driver. This factor pre-adjusts or sets the "central switching mechanism" to favor certain trunk lines.

The engineer may sometimes forget that, since he is familiar with the layout of a design, he represents such a pre-set system, and consequently he reacts more easily and quickly than does the strange driver who is not thus set. Sometimes the engineer, quite unintentionally, by means of signing, or by means of the preceding alinement of the highway, may even give the strange driver what corresponds to the "wrong number," thus leading him to expect a different type of layout from what he is actually approaching. This results in the "surprises" mentioned by the author. Under these circumstances, it takes extra time for the human mental "switching system" to unscramble the wrong numbers and redial the right ones.

To avoid this confusion, it is important that the engineer submit his new design or redesign to tests from the point of view of the every-day driver. He can do this in two ways. First, he can try to put himself in the place of the strange driver. This is somewhat difficult to do, but after some practice and with the help of friends who are not familiar with the design he may be able to make a fair guess at the stranger's state of mind as he approaches the location. Second, he can measure or observe a large number of drivers on the new design before it is actually installed. This can be done by means of temporary installations, with sandbags or with markers on movable stanchions. It is also possible, for some problems, to select certain test locations which involve similar features, and to make measurements of traffic behavior on these.

From such studies, there will result not only proper design, but also the most effective wording of warning signs and placement of warnings.

5. For any installation on a moderately high-speed route, it is very important that the placement of warnings allows the motorist sufficient time. This time must include not only his own "perception-and-judgment-time" switching operations, but also comfortable deceleration to a speed slow enough for the maneuver.

A time period longer than that discussed herein under Section 3 is thus required, as has also been recognized by the author and by other leading design engineers.[40]

Again, to be specific: In a recent study[38] it was found that perception-and-judgment time, where a judgment of complex speeds was involved, required at least 2.8 sec for the motorist to make up his mind. This value was selected to include about 80% of the motorists. Measurements have not been made at intersections, but a similar judgment-time value probably holds. To this must be added time for deceleration. John Beakey[41] showed in 1938 that as motorists approached a stop sign and decelerated from 50 or 60 miles an hr to 30 miles an hr, they required on the average about 8 to 9 sec. Slowing to 20 miles an hr required from 10 to 11 sec on the average. On this basis, it is apparent that if the installation requires slowing to a turn-off speed of 30 miles an hr, a total of about 12 sec of warning is required, whereas for a turn-off speed of 20 miles an hr to be reached, a warning time of 13 to 14 sec would be needed.

Since the vehicle is decelerating through this distance, a warning time of about 10 sec and 12 sec, respectively, is probably sufficient when converting warning time into distance (for sign design) in terms of the design speed. The deceleration distances of Mr. Beakey's study bear out this conclusion.

From studies of the effect on the motorist of different types of signs,[42] it is possible to predict the necessary height, width, and type of reflectorization of lettered signs to give sufficient warning.

Summary.—1. The author has been commended for his discussion of the human factor in relation to channelization. It has been pointed out that the human factors involved could be stressed even more in the discussion of channelization and traffic design, since they are, in the final analysis, the fundamental cause of the problem which is being treated. However, at the same time it should be emphasized that they are also the fundamental source and reason for the existence of this automobile traffic for which the highways and streets are being designed.

2. The human factors treated by Mr. Kelcey have been recognized by many leading design and traffic engineers, but they have been too often thought of as nebulous and impossible of engineering treatment. It has been shown that this is not the case, and that if the proper approach is made by sound physical research based on psychological and physiological knowledge, these factors can be made definite and specific in a practical way for design purposes. That is,

[40] "Needed Research for the Determination of Sight Distance at Intersections," by Joseph Barnett, *Proceedings*, Highway Research Board, 18, 1938, p. 76.

[41] "Acceleration and Deceleration Characteristics of Private Passenger Vehicles," by John Beakey, *loc. cit.*, p. 81.

[42] "Legibility Distance of Highway Destination Signs in Relation to Letter Height, Letter Width and Reflectorization," by T. W. Forbes and R. S. Holmes, Jun. Am. Soc. C. E., *loc. cit.*, 19, 1939, in press.

certain minimum and limiting values can be obtained for the guidance of designers.

3. Certain illustrations have been given of definite time values, and corresponding distance values resulting therefrom, which may be considered as the minimum values for traffic design and channelization problems. The studies from which these were derived have been cited.

JAMES S. BIXBY,[43] ESQ. (by letter).[43a]—To Mr. Kelcey's able exposition of the advantages derived from vehicle traffic channelization there should be added the warning that every raised island or obstruction provided for vehicle control within the traveled-way is a potential menace to traffic, making it necessary frequently to weigh the advantages of channelization against the hazards that will be introduced by islands in traffic.

Vehicle drivers tend to run straight ahead at the maximum practical speed and any obstructions intermittently introduced in this straight-ahead path are certain to involve danger, in spite of warnings which may seem to be adequate.

At the intersection of a main route and secondary road, islands in the throat of the intersection may be installed and maintained successfully on the secondary road where normal car operation requires the slowing of traffic, whereas the same islands placed on the main road may be the cause of serious trouble, in spite of flashing signals and other safety measures. For the reasons stated, cities and urban areas are more suited to island traffic control because of better lighting and slower traffic, but it is a matter of record that almost every city in the United States has had some experience with traffic-control obstructions that have been installed and later abandoned.

At locations where island control of traffic has proved dangerous, the difficulty has been solved by removing the islands and substituting pavements of a different color from those surrounding the islands. The flat areas of different color, of course, do not provide the same positive control as raised islands but they do develop a very positive influence in controlling the streams of traffic without danger to the vehicles involved.

As suggested by Mr. Kelcey, grade separations are better than traffic circles because the circle, after all, is an obstruction in the path of normal straight-ahead traffic and, therefore, involves danger even if traffic is warned and controlled. For traffic circles in rural areas, adequate entrance drives should be provided so that the speed of main line traffic need not be too sharply curtailed in order to enter the rotary area.

If traffic circles are to be undertaken as a step toward ultimate separation of grades, due consideration must be given to the fact that a circle more than 300 ft in diameter, with good approach drives, can be built on about five acres of land, whereas a separation structure with access drives in each quadrant may require from ten to twenty-five acres, depending on the minimum radius of curvature desired on the access drives. On many traffic-circle installations, the development of business immediately adjacent to the circle will make the cost of later developing a grade separation prohibitive.

---

[43] Dist. Engr., Dept. of Public Works, State of New York, Poughkeepsie, N. Y.

[43a] Received by the Secretary April 5, 1940.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

TREND IN HYDRAULIC TURBINE PRACTICE
A SYMPOSIUM

Discussion

By Messrs. Paul L. Heslop, and J. D. Scoville

Paul L. Heslop,[33] M. Am. Soc. C. E. (by letter).[33a]—Model testing has been used extensively as a means of advancing the art of turbine building, but it also has an immediate and practical side. When a customer buys a turbine he wants to be sure that it will perform as anticipated, and model testing has now reached the stage of reliability where reasonably accurate forecasts of performance should be made. Several late government contracts for large wheels, of which Bonneville is an example, have specified that acceptance shall be based on stepped-up model tests. This applies particularly to large low-head developments where accurate measurement of flow is difficult for the reasons stated by Mr. Davis.

At Bonneville a very thorough field test was made on the 81-in. Kaplan station-service unit in order to prove or disprove the reliability of the forecasting value of model tests. The tests show that, throughout the entire range of performance over a wide variation of load and of head, actual results do not differ more than 2% from values that could be predicted and, in the vicinity of contract head and rating, the agreement is a matter of tenths of a per cent.

With the field data at hand, some paper experimenting was done with step-up procedures. An expectation curve is shown in Fig. 22. To derive this curve the model efficiencies were stepped up by Moody differentials. The power was first stepped up by the relation of three-halves power of the diameters and then further increased by application of the same Moody differentials. The agreement around specified head and power is interesting. In computing the Moody differentials, the peak efficiencies of the model without respect to speed were used, which is the conventional method. Had the speed at 1-ft head of the model corresponding to actual prototype speed been used for effi-

Note.—This Symposium was published in November, 1939, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings*, as follows: January, 1940, by Messrs. W. S. Pardoe, and Donald H. Mattern; March, 1940, by Messrs. Lewis F. Moody, and R. E. B. Sharp; and April, 1940, by Messrs. Martin A. Mason, and E. Shaw Cole.

[33] Civ. and Hydr. Engr., Portland, Ore.

[33a] Received by the Secretary April 4, 1940.

ciencies, the agreement between predicted and field tests would have been even closer.

One of the means used on the Bonneville runners to combat cavitation was to cut out, and pre-weld with stainless steel, certain blade areas most susceptible to cavitation. In making cavitation experiments on a composite blade of this kind, it should be borne in mind that paint will not adhere as well to stainless steel as to cast steel.

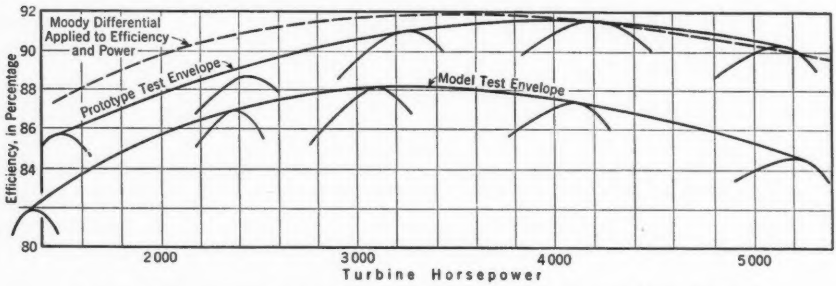


FIG. 22.—BONNEVILLE STATION-SERVICE TURBINE; 81-IN. KAPLAN TYPE TURBINE UNDER A 50-FT CONTRACT HEAD

One purpose of the transparent models used in the Bonneville experiments was to decide upon a workable scroll case in the shortest possible time. When Bonneville was authorized, it was desired to put the greatest number of men to work in the shortest possible time. Immediate work on power-house construction could further this aim. At a later date, cavitation and performance tests were run on models by the turbine manufacturer. Time permitting, perhaps the ideal situation would be to combine visual analysis with the performance and cavitation model tests, using transparent sides for model make-up.

J. D. SCOVILLE,[34] ESQ. (by letter).[34a]—Mr. Winter discusses the tendency toward the use of a horizontal splitter in draft tubes. Fig. 23 is a section through the Bonneville power house showing the shape of the intake, scroll, and draft tube. The horizontal splitter shown gave a substantial improvement in draft tube performance as compared with tubes without splitters when tested with a model turbine. This type of tube was used also on the station service unit at Bonneville and partly explains the excellent performance of the turbine, which is shown in Fig. 3.

Scroll.—Model tests indicate that a simple design of scroll is superior to that which was used in the Bonneville power house, the plan of which is shown in Fig. 24. It will be noted that there is a multiplicity of piers and islands, the purpose of which is to guide the water into the turbine without the formation of eddies. It would appear that the additional skin friction introduced by the extra pier area more than compensates for any possible reduction of eddy loss. Fig. 25 shows the comparison of model tests on the U. S. Engineer's design of scroll in conjunction with a 16-in. model runner homologous to the

[34] Asst. Chf. Engr., S. Morgan Smith Co., York, Pa.

[34a] Received by the Secretary April 2, 1940.

Bonneville turbine. These comparisons are made at model speeds corresponding to 50-ft and 60-ft heads and show that the simple design of scroll is superior to the more complicated one which was used. Structural considerations made

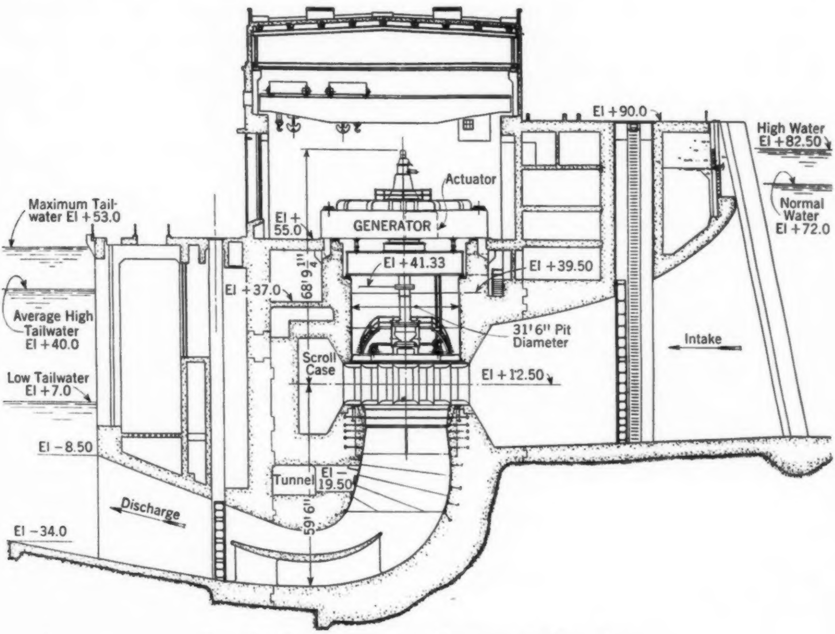


FIG. 23.—SECTION THROUGH BONNEVILLE MAIN UNIT

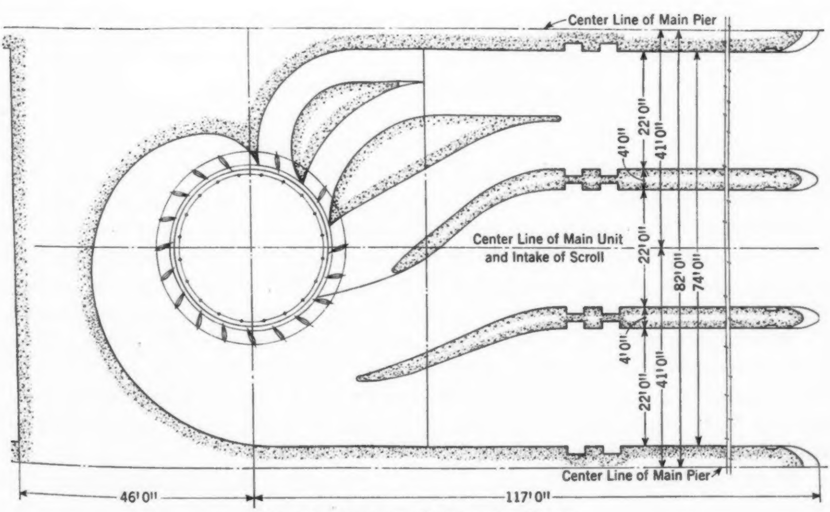


FIG. 24.—PLAN OF BONNEVILLE SCROLL

it necessary to use the U. S. Engineer's design. Where it is not necessary to carry the intermediate piers too close to the speed ring of the turbine, the simpler design is preferable.

Scroll Velocities.—Mr. Winter states that it is possible to use considerably higher scroll velocities with improved turbine performance. The writer has tested plate steel scrolls with velocities 60% above the normal value and found a loss in efficiency of about 1%. This would indicate the possibility of using higher scroll velocities. However, he questions the advisability of going to so high a velocity because, although there may be little loss when the unit is new, the loss may be substantially greater when the unit is several years old and the scroll is roughened from corrosion.

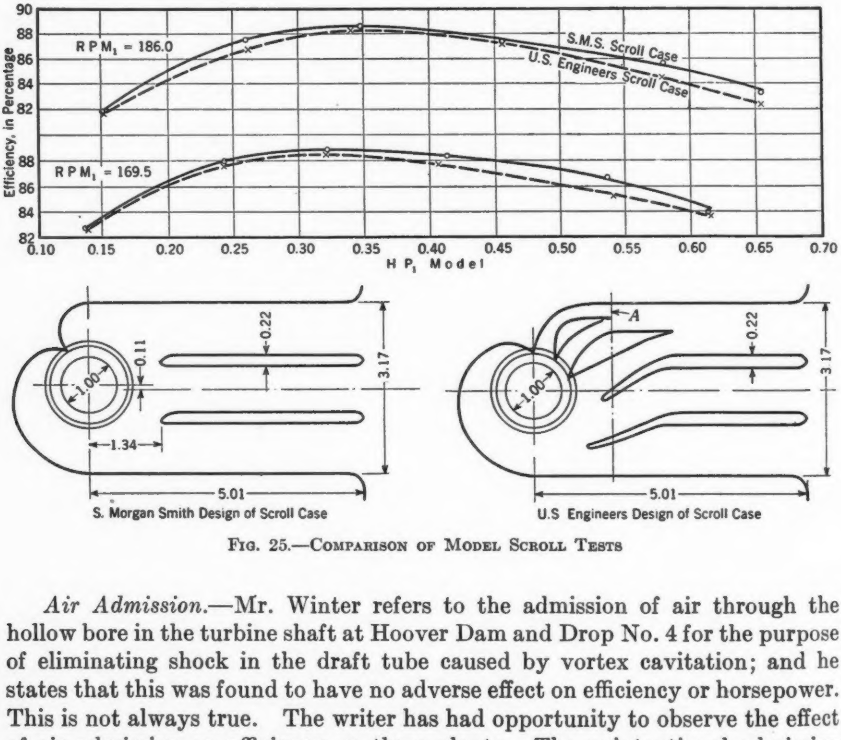


FIG. 25.—COMPARISON OF MODEL SCROLL TESTS

Air Admission.—Mr. Winter refers to the admission of air through the hollow bore in the turbine shaft at Hoover Dam and Drop No. 4 for the purpose of eliminating shock in the draft tube caused by vortex cavitation; and he states that this was found to have no adverse effect on efficiency or horsepower. This is not always true. The writer has had opportunity to observe the effect of air admission on efficiency on three plants. The unintentional admission of air through the hollow shaft of a 53,000-hp unit under a 135-ft head caused a loss in efficiency of 3%. Experimental air admission to a 42,000-hp turbine under a 104-ft head caused an efficiency loss of 3% at the peak and a gain of 1% at 25% load. Air leakage through the packing gland of a 3,000-hp turbine under a 54-ft head produced an efficiency loss of 4.5%. The advisability of admitting air must be determined for each installation.

Moody Differential.—It is the writer's experience that he obtains values higher than the computed Moody differential on the smaller sizes of runners,

to a diameter of 100 in. Beyond this diameter there is some indication that the Moody step-up ratio is a little too high; but this possibility is supported by very meager data. The writer agrees with Mr. Davis that more tests on very large units are desirable. Only by numerous points on the curve can it be determined whether the Moody exponent should be 0.25 or 0.20.

Bonneville Tests.—In 1938 power tests were made on the main units at Bonneville, and efficiency tests were made on the service unit. The main units, two in number, have a rating of 60,000 hp under a 50-ft head. Before the job was built, complete model tests were made in the manufacturer's laboratory and at Holtwood, Pa. Field tests at four heads, from 29 to 53 ft, showed about 8% more power than that proportioned from the model tests. This increase in power is certainly evidence of a substantial increase in efficiency, although it is not necessarily of the same amount. Both of these units were operated successfully at 80,000 hp, or 33% above the guarantee. Cavitation was noticeable at this load and it would not be advisable to run continuously at this output.

The station-service unit has a guaranteed capacity of 5,000 hp and is a Kaplan turbine. The efficiency tests showed a maximum efficiency of 93%. This value of efficiency is in agreement with the expected performance as stepped up from model tests by the Moody formula. At higher loads the efficiency was 1% above the computed value and at lower loads 1% below.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

ANALYSIS OF LEGAL CONCEPTS OF SUBFLOW AND PERCOLATING WATERS

Discussion

BY MESSRS. EDWARD F. TREADWELL, O. E. MEINZER, M. R. LEWIS, AND BAYARD F. SNOW

EDWARD F. TREADWELL,[58] ESQ. (by letter).[58a]—The writer does not presume to discuss the principles of hydrology set forth in this paper. Of course, both lawyers and courts must sometimes consider and discuss such principles, and they recognize the difficulty of speaking on that subject with such accuracy as meets the exacting demands of science or scientists. The paper may be of assistance in aiding the legal profession in understanding the principles of hydrology applicable to underground water and in stating them with accuracy.

At least half of the paper is devoted to criticisms of judicial decisions in cases involving some of those principles. As to that portion of the paper one may presume to offer some comment.

Practically every case cited is attacked as being in some way in violation of the positive rules of hydrology developed by the paper. This probably indicates that the scientist has as much difficulty in understanding a judicial decision as the lawyer has in stating a scientific principle. Such a paper would be of much more value if it had been prepared in cooperation with a lawyer. The writer disagrees entirely with the criticisms made of the decisions in question.

The first case criticized is Maricopa County Municipal Water Conservation District No. 1 vs. Southwest Cotton Company[20] involving the underground water of Salt River Valley. In that case the trial court apparently held that the plaintiff had certain rights on the "theory" that the underground water was water of an underground watercourse. The authors of the paper seem to complain that the trial judge properly held conclusive the "geological evidence," and that the Supreme Court disregarded the geological or scientific evidence; and they lament that "if courts refuse to accept scientific evidence indicating

NOTE.—This paper by C. F. Tolman and Amy C. Stipp was published in December, 1939, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: February, 1940, by Donald M. Baker, M. Am. Soc. C. E.; and April, 1940, by Messrs. Samuel C. Wiel, Hyde Forbes, and Ronald B. Harris.

[58] Treadwell & Laughlin, San Francisco, Calif.

[58a] Received by the Secretary March 4, 1940.

[20] 39 Ariz. 65, 367, 4 Pac. (2) 369 (1931), 7 Pac. (2) 254 (1932).

natural conditions their decisions may not be well founded." As a matter of fact, the Supreme Court did not refuse to follow scientific "evidence," but held that there was nothing more than scientific or geological theory of channels somewhere in Salt River Valley, but no evidence as to where these underground channels were located. It even accepted the "theory" of such channels, but still held there was no "evidence" of their location. This does not seem to be subject to criticism. Differences arise even as to the existence of surface streams, and different courts have come to different conclusions on the same evidence. It is not strange that different conclusions are reached as to underground channels. Geological principles may be as positive as the authors indicate; still the courts need not follow every witness that attempts to establish the "facts" to which such principles are to be applied. A principle of geology can no more be applied than can a principle of law, until the facts are determined. The court does not violate the principle in either case because it refuses to apply it on the ground that the necessary facts have not been established.

The next case criticized is Vineland Irrigation District vs. Azusa Irrigation Company.[21] In this case the Supreme Court was not stating what occurs under certain conditions, but what the trial court found actually occurred. The trial court found, as a fact, that the taking of the water creates an artificial draft upon the surface flow of the stream, draws down a part of it, and weakens and injures the natural bed of the stream, and tends to interrupt and carry away the surface flow. The trial court also found that the stream "saturated" the gravelly bed of the stream. Still the authors of the paper misconstrue this decision into holding that an influent flow generally supports the surface flow. In fact, the condition involved was exactly what the authors define as "an influent subflow, with ground-water mound in contact with surface flow." This must be so if the entire bed was saturated. The decision, therefore, seems proper.

The next case apparently criticized is Lemm vs. Rutherford.[22] In that case the right to construct a sump into which the seepage from a nearby ditch was collected and from which it was taken was involved. The court was careful to point out that there was no independent body of subsurface water. The only source of underground water was the seepage of the water from the ditch. The intimation of the paper would seem to be that the seepage in no way supported the water in the ditch and, therefore, that the landowners should be permitted to take it. Of course, all earthen ditches seep and the amount of the seepage is due to the porosity of the material through which they pass. Even a porous material is some support for the water in the ditch; otherwise all the water would seep from the ditch. The water that fills the pores of the material also acts as a support. If that support is taken away by a trench or sump near the ditch from which trench or sump water is pumped, then there would seem to be room for the claim that such artificial works induced seepage and thus took away the right of natural support. The writer can see no reason why this could not occur.

---

[21] Vineland Irrigation District vs. Azusa Irrigation Co., 126 Cal. 486, 58 Pac. 1057, 46 L.R.A. 820 (1899).

[22] Lemm vs. Rutherford, 76 Cal. App. 455, 245 Pac. 225 (1926).

The next case criticized is City of San Bernardino vs. City of Riverside.[23] Everything stated by the court in that case is based upon the assumption that the stream runs over porous material saturated with water, and that the underflow water supports the stream, either by upward or lateral pressure, or feeds it directly. The authors misinterpret this into the "concept that 'support' is generally afforded the surface stream by subflow," which concept they state is erroneous. Here again they overlook the finding that the material was "saturated," which brings the subsurface water in contact with the surface stream, which condition the authors themselves admit does support the stream.

The writer is entirely unable to understand wherein any of these cases violates any positive rule of hydrology, even if he were to admit the correctness of all the rules stated by the authors. As stated, the writer does not presume to question any of those rules, but he would like to read further evidence to support the positive rule announced by the authors that as long as there is a column of unsaturated material between the surface stream and the underground water table, a lowering of the water table will not affect the surface stream. May the writer be so bold as to suggest that the water table is at all times supplying this column with water, by capillarity, for several feet above the water table. The lowering of this table reduces that supply, and this leaves just that many more voids to be filled by the seepage from the surface stream. Irrespective of whether or not such facts would make the taking of water from the underground body unlawful, the writer believes that these considerations might tend to weaken the absolute rule stated by the authors.

O. E. Meinzer,[59] Esq. (by letter).[59a]—In showing clearly the confused and erroneous character of many of the legal concepts relating to ground waters, and in substantiating previous discussions of this important subject, the authors have rendered a valuable service. The paper is especially timely because of the great interest, in recent years, in ground water as a source of water supply and of the more general appreciation of the fact that this resource is not inexhaustible. The treatment of the subject relating to ground water occurring under water-table conditions is so effective that it is regretted that the paper was limited in scope so as not to include a discussion of the equally important subject of ground water under artesian conditions.

It is noted that the authors make no reference to the paper by D. G. Thompson and A. G. Fiedler, Assoc. M. Am. Soc. C. E., on "Some Problems Relating to the Legal Control of the Use of Ground Waters," which was awarded the John M. Goodell prize of the American Water Works Association for the most meritorious paper published in its *Journal* during 1938.[60] This paper discusses some of the fundamental principles of ground-water hydrology which should be understood in any attempt to outline a sound doctrine of legal control of the use of ground water. In a discussion of the paper by Harold Conkling, M. Am.

---

[23] 186 Cal. 7, 198 Pac. 784 (1921).

[59] Geologist in Charge, Div. of Ground Water, Geological Survey, U. S. Dept. of the Interior, Washington, D. C.

[59a] Received by the Secretary March 18, 1940.

[60] *Journal*, Am. Water Works Association, Vol. 30, 1938, pp. 1049–1091.

Soc. C. E., referred to by the authors, Mr. Thompson[61] has also pointed out, by discussion of cases that have already been decided and of conditions observed in other parts of the country where litigation has not yet arisen, the inconsistencies that result from the unscientific classification of ground waters that have crept into the law.

The erroneous legal concepts regarding ground waters were first presented to the courts at a time when true scientific concepts had not yet been well developed and doubtless largely by persons who were not familiar with the knowledge of the subject that was then available. These erroneous concepts have been perpetuated because of the reluctance of the courts to depart radically from precedents established in previous cases. The matter of precedent confronted the framers of the New Mexico ground-water law enacted several years ago, which declared ground water to be subject to appropriation in that state. As a result of an intensive investigation of the Roswell artesian basin by the U. S. Geological Survey, in cooperation with the state engineer of New Mexico, the scientific facts regarding the occurrence of the ground waters of that area were thoroughly determined. Nevertheless, recognizing the strong influence that the precedent of earlier decisions has upon the courts, the framers of the law were loath to disregard the more or less generally accepted legal terminology that had been supported by earlier decisions. Therefore, the law was made to apply to "the waters of underground streams, channels, artesian basins, reservoirs, or lakes, having reasonably ascertainable boundaries." It is understood that this terminology was intended to be sufficiently broad to include the so-called "percolating water" under the doctrine of appropriation. Such a broad interpretation would seem to be justified because all bodies of ground water can be included under one or more of the terms "streams, channels, artesian basins, reservoirs, or lakes" and the boundaries of virtually all bodies of ground water can now be determined with reasonable accuracy.

The authors discuss "Effects on Surface Flow of Pumping from Influent and Effluent Subflow"; they do not discuss the converse subject which might be stated "Effects on Subflow of Diversions from Influent and Effluent Surface Streams." In most cases it is the user of ground water who has been on the defensive; and unless the evidence has been clear that there was no interference, the case has generally been decided in favor of the user of the stream water. However, if the ground water constitutes a part of the stream before it appears as surface water, does not that same ground water also constitute a part of the stream for purposes of appropriation? Accordingly, if the person who pumps subflow is required to desist in favor of a prior appropriator of surface flow, should he not in turn be given priority over a subsequent appropriator of the surface flow? Questions of this kind have arisen in the Mokelumne River area, in California, and in the Platte River Valley, in Nebraska, where heavy pumping from wells was practiced before the diversions from the rivers were made. A careful consideration of all factors involved must lead to the conclusion that eventually, in many areas, the surface water rights—whether by

[61] Discussion by D. G. Thompson of "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 102 (1937), pp. 798–819 (especially pp. 803–811).

court adjudication or by appropriation—must be evaluated in relation to the rights to use ground water, and vice versa.

M. R. Lewis,[62] M. Am. Soc. C. E. (by letter).[62a]—The difficulties, to which the arbitrary and artificial classification, by the courts, of ground water into "subsurface streams" and "percolating waters" has led, are well presented in this paper. It is to be hoped that as the trend toward state control of the development and use of ground water proceeds, legislatures and courts will be guided by a genuine understanding of ground-water hydrology.

In Part I, under the heading "Effects on Surface Flow of Pumping from Influent and Effluent Subflow," the authors state that, "Regardless of whether influent or effluent conditions existed prior to the development of such a cone of depression below stream bed, the pumping well is supplied by diversion of subflow plus water furnished by the induced influent seepage from the surface stream." This statement should be qualified to the extent of excluding those influent streams where the water table is not in contact with the surface stream. In general, further lowering of a water table that has already lost contact with a surface stream will not increase the stream loss. This point is correctly stated in the paper under "Subsurface Water Course: Influent and Effluent Conditions."

In the case of a subsurface stream whose banks are permeable, but less so than the channel deposits, the writer does not agree with the statement that,

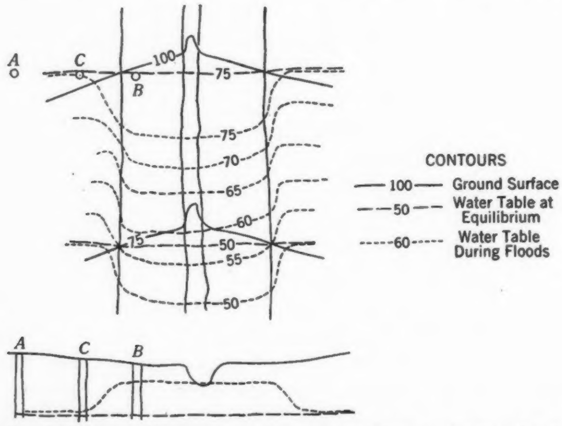


Fig. 11.—Water-Table Contour Pattern in a Subsurface Channel Having Less Pervious Banks

"When flood flow produces a pronounced mound in the subsurface stream, the water-table contours beyond the subsurface banks are also bent into a less pronounced influent shape, indicating slight influent movement from subflow into the older valley fill" (see Part I, heading "Subflow Indicated by Contour Pattern").

It is believed that the shape of the contour close to the subsurface bank in the less pervious material will be more, rather than less, pronouncedly influent.

[62] Senior Agri. Engr., SCS and Oregon Agri. Experiment Station, Corvallis, Ore.

[62a] Received by the Secretary March 18, 1940

From Fig. 11 it will be evident that water-table contours plotted from readings in wells *B* and *A* alone would indicate a flatter slope of the water table outside than inside the old channel, whereas readings in *B* and *C* would show the much steeper slope to be expected in the more impervious material. As is inferred in the sentence quoted, the movement of water into the less permeable banks will be less rapid than in the channel deposit, but this will be true because of the decreased permeability and in spite of a steeper water-table gradient.

Similarly in the case of the trench in the subflow channel indicated in Fig. 10, the contours just outside of the subsurface channel should be more nearly parallel to the bank than the contours inside the channel.

To enable the courts to consider the general applications of ground-water hydrology and law when they are called upon to decide specific cases, it would be well if appropriate state or federal agencies were permitted and empowered to appear as "friends of the court" in order to present such general applications, and thus protect the general public from the ill effects that might result from decisions based on incorrect or incomplete presentation of facts or faulty reasoning.

The authors note that no recent text on water-right law is available. It should be noted that Wells A. Hutchins, irrigation economist, Division of Irrigation, Soil Conservation Service, has prepared such a text.[63]

BAYARD F. SNOW,[64] M. AM. SOC. C. E. (by letter).[64a]—Analyzing erroneous concepts of underground hydrology is no simple task, and the authors deserve commendation for the able manner in which they have presented a number of examples of ground-water flow. There is no doubt that many legal decisions have been based upon insufficient or misleading engineering assistance and have in turn led to lines of questioning which have made it difficult, if not impossible, for competent witnesses to present the truth to the court. Notable in this respect is the decision[65] which established in England the principle that has been largely followed in the United States—namely, that water percolating through the soil and not flowing regularly or definitely is, while there, a part of the land and completely subject to the use and control of the owner. If this paper can serve as a foundation for decisions allocating rights to ground waters, the relationship between law and justice can be closer and a forward step can be taken in the conservation of one of the most valuable and least understood of the natural resources.

There is, perhaps, a danger that in the simplified examples given a reader, particularly if looking for an explanation to fit his conditions, may overlook certain fundamental facts. Removal of part of the runoff, whether from a well or from a surface stream, will diminish the total flow from a watershed by the amount so removed. The effect of such removal may be felt, immediately below, entirely in the ground-water body or in the surface stream, or in part

[63] "Selected Problems in the Law of Water Rights in the West," by Wells A. Hutchins, U. S. Dept. of Agriculture (publication pending).

[64] Cons. Engr.; Pres., X. Henry Goodnough, Inc., Boston, Mass.

[64a] Received by the Secretary March 21, 1940.

[65] Acton vs. Blundell, 12 M. and W. 324.

by each; but farther downstream the effect is almost certain to be divided between the two.

Under natural conditions, surface streams are maintained and augmented by effluent flow of ground water, generally gaining in volume as they progress toward the ocean; but, at places in their courses where ground water is lower than surface water level, they may lose volume to such extent as the permeability of the bed and banks permits. The relationship between rainfall, evaporation, and transpiration will determine the changes in volume of the total runoff; the cross-sectional area, permeability, and slope of the aquifer will fix its hydraulic capacity and consequently the "water table" for a given volume, subject to the influence of strata of less permeable material. These less permeable strata may result in a perched water table or in an artesian condition.

The authors have called attention to the disagreement between legal and scientific definition of "percolating waters." The definition of "tributary percolating waters" described by Kinney[12, 14] (see heading "'Percolating Waters': Subsurface Reservoir") would deny to mankind the use of "percolating waters," since waters which do not "form a vast mass of water * * * moving slowly down to lower levels," are not available for pumping or diversion as the authors state.

The law has divided available ground water into two classifications and has differentiated between the rights which may be acquired in accordance with that classification; but if the cited definition is to govern, rights to "percolating waters" are of no practical value. The authors say that " 'subsurface streams,' 'percolating waters,' and 'artesian waters,' as defined by lawyers, all move by percolation in interconnected openings of capillary size." Exception might be made in the case of cavernous rock channels forming "subsurface streams," but these are not under consideration at the moment. Available ground waters are percolating waters, and the writer's experience has made him very reluctant to answer legal questions about underground streams. Beds and banks of freely permeable aquifers may be relatively impervious or may be only slightly less permeable than the body of the stream. Coarse, water-bearing deposits of gravel and sand frequently disclose no connecting link of permeable material, no matter how extensive the underground exploration may be. Such connecting links between aquifers which appear to be definitely separated, either horizontally or vertically, by almost impervious deposits can be found to exist by pumping tests, and, if confined in cross-sectional area, may be "subsurface streams" of relatively large hydraulic capacity; but response of an observation well to a distant pumping well does not in itself warrant the conclusion that such a stream, with definite "bed and banks," exists. On the other hand, the connection may be large in cross section and without definite boundaries but gradually merging with a less pervious material. In such cases the "bed and banks" do not exist. In either case the entire cross section of the ground-water body is similar, in many respects, to a surface stream in which the velocity decreases markedly

[12] "Treatise on the Law of Irrigation," 2d Ed., by C. S. Kinney, Bender-Moss Co., San Francisco, 1912, pp. 2169–2170.

[14] 156 Cal. 603, 105 Pac. 755 (1909).

as one approaches the banks. If such a stream is partly obstructed by vegetation or porous materials, velocities will be greatly reduced, particularly through the obstructed part of the section. The flowing waters, however, whether in the less obstructed channel or among the weeds and obstructions near the shore, are all part of the surface water body. One can scarcely believe that a court could differentiate between the more freely flowing and the more obstructed waters in allocating rights for diversion.

In the New England area, at least, the coarser glacial deposits are frequently found to be in random lenses or pockets, and the relatively large and permeable potential aquifers are separated, horizontally and vertically, by finer and less permeable materials. If natural conditions produce little differential head, communication between two such deposits will not be induced to any marked degree; but if a considerable head is created by pumping, the flow between the two permeable deposits by what was originally a vein only slightly more pervious than surrounding material will be developed so that it will tend to become an "underground stream," the bed of which may be made relatively impervious by the addition of the finer grains moved from the interstitial spaces in the cross section and redeposited in interstices in the bed. The legal definition of "underground stream," however, takes into account the permanence of the channel, and it appears that states which have distinguished between "percolating waters" and "underground streams," with respect to prescriptive rights, have applied to the latter, when in a "distinct, permanent and well defined channel," the same rules as apply to surface streams. If permanence is essential to the legal existence of an underground stream, and if the disturbance of natural conditions attendant on the development of a well tends to destroy evidence as to whether any such stream did in fact previously exist, or even creates such a stream or concentration of flow, proof of that legal existence may be difficult. Wells alongside of surface streams have drawn water which could conclusively be shown to be surface water although the surface stream had bed and banks of fine, tight, water-resisting material. In other cases, wells similarly located drew only ground water. Except for the physical and chemical characteristics, the origin of the well water could not be determined. Similarly, an underground stream, flowing by a "defined and known channel" (using "defined" in the legal sense of "a contracted and bounded channel, although the course of the stream may be undefined by human knowledge"), may or may not contribute to the yield of a well, and differentiation of rights by the legal classification of underground waters is impracticable.

Although "all underground waters are presumed to be percolating, and to take them out of the rule with respect to such waters the existence and course of a permanent channel must be clearly shown," the distinction is one of degree and is not absolute; and the writer agrees wholeheartedly with the authors that "A few general rules of law founded on well-established hydraulic engineering principles should eliminate much of the confusion that has arisen in the attempt to follow prior legal rulings based on an arbitrary classification not in agreement with occurrence of water below ground surface." The foregoing can be illustrated by reference to court decisions in at least eight states: Florida,[66]

[66] Tampa Waterworks Co. vs. Cline, 37 Fla. 586, 20 So. 780, 53 Am. St. Rep. 262, 33 L. R. A. 376.

Iowa,[67] Maryland,[68] Mississippi,[69] Ohio,[70] Oregon,[71] West Virginia,[72] and Wisconsin.[73]

The writer recalls litigation involving legality of a taking of a well site under an old charter which granted right to take "* * * the water of any stream or streams, or of any spring or springs * * *." After tests by 2.5-in. driven wells, the taking was made and a gravel-packed well constructed. The plaintiff contended that the well did not take waters of either stream or spring. It was obvious that the well did not take water from a stream or spring, whether one includes underground streams or not, and also whether those words are as understood by the layman or have any special meaning in the profession or trade. However, for two reasons it appears that the water supply was within the limits set by the charter. First, the wording "stream or streams, spring or springs" was evidently intended for what, in later years, would have been described as any sources of surface or ground water. Second, any available ground water is water in motion toward a point of discharge at a lower level, which point of discharge is a spring, so that ground water naturally is the water of a spring or springs.

Another result of legal misunderstanding of ground-water hydrology and of legal precedents that have been set up (and a very serious one) is illustrated in decisions which may be summarized as follows: Connecticut,[74] New York,[75] Michigan,[76] and Kentucky.[77,78]

Wells and underground waters may be contaminated by disposal of gas house wastes, oil, and other substances deposited in or on the ground in such a way that they leach through the earth and pollute the ground-water body. In some jurisdictions no relief from such contamination was granted, inasmuch as intent to injure or negligence could not be proved. One judicial comment indicates a tendency to be guided by public policy rather than legal theory, basis for decision which should appeal to engineers only if the public policy is well founded and recognizes the importance of ground water and the conditions which will affect its quantity or quality and any future needs. In another case cited, the right to destroy a well by withdrawing water from adjoining land is taken as precedent for a right to corrupt the waters, since the injury "is the same kind and degree in the two cases." In all of these cases the decision took into account the legal classifications of ground waters.

It is obvious that although a few decisions have recognized that all underground waters are presumed to be percolating, in general, the courts have distinguished between percolating waters and underground streams. In the

[67] Barclay vs. Abraham, 121 Iowa 619, 96 N. W. 1080, 100 Am. St. Rep. 365, 64 L. R. A. 255.

[68] Western Maryland R. Co. vs. Martin, 110 Md. 554, 73 Atl. 267.

[69] Clarke County vs. Mississippi Lumber Co., 80 Miss. 535, 31 So. 905.

[70] Wyandot Club vs. Sells, 9 Ohio S. and C. Pl. Dec. 106, 6 Ohio N. P. 64.

[71] Taylor vs. Welch, 6 Oreg. 198.

[72] Pence vs. Carney, 58 W. Va. 296, 52 S. E. 702, 112 Am. St. Rep. 963, 6 L. R. A. N. S. 266.

[73] Huber vs. Merkel, 117 Wis. 355, 94 N. W. 354 (see 48 Central Digest title "Waters and Watercourses," No. 109).

[74] Brown vs. Illius, 27 Conn., 84, 71 Am. Dec. 49.

[75] Dillon vs. Acme Oil Co., 48 Hun. 565, 2 NYS 289, 291.

[76] Upjohn vs. Richland Board of Health, 46 Mich. 542, 9 N. W. 845, 848, 41 Am. Rep. 178.

[77] Kinnaird vs. Standard Oil Co., 89 Ky. 468, 12 S. W. 937, 7 L. R. A. 451, 25 Am. St. Rep. 545.

[78] Long vs. Louisville and Nashville R. Co., 128 Ky. 26, 107 S. W. 203, 13 L. R. A. (N. S.) 1063, 16 Am. Cas. 673.

normal course, these decisions will be precedent to more and more opinions, all based upon an erroneous concept, a distinction which has existed in the minds of men but not in the forces of nature.

The fundamental necessity of water to all living beings makes it important that ground-water resources be understood not only by those who use them and their technical advisers, but also by those who fix the rights to their use. It is important that ground-water hydrology be better known in order that rights to such water may properly be allocated among owners on any ground-water body. It is even more important, however, to prevent the loss of that ground-water body to man by its contamination. A misunderstanding which results in unjust allocation of water may affect only two owners, and that only during the period of unjust diversion. Other owners farther downstream may still have an adequate supply for their needs. There may even be adequate supply for the disputing owners at some additional expense. The problem, in short, is whether one or the other shall utilize the available water without interference. In such a problem the public has no basic interest. On the other hand, a misunderstanding of ground-water flow which results in legal permission for the contamination of the ground-water body and of the earth through which it percolates not only affects the adjoining owner but may destroy the value of the ground-water body throughout its entire course, and even for years after the contamination is discontinued, although that body might otherwise have been an adequate and satisfactory source for many users.

Contrary to the foregoing opinion, withdrawal of water from an excavation on adjoining lands is unlikely to destroy a well, at least beyond remedy, except by interception of what would legally be an underground stream or of substantially the entire ground-water body, whereas contamination of the ground water can certainly destroy the neighbor's well beyond remedy. The writer believes that the implications of the decisions permitting contamination of percolating waters are far more serious and inimical to the public good than any allocation of those waters, no matter how erroneous its basis, to what presumably is a purpose useful to mankind.

The authors have called attention to the paper by Mr. Conkling.[3] It would be well for any person interested in the legal phases of ground water to review that excellent paper and its discussion. It may be significant that both the present paper and that by Mr. Conkling came from California and that most of the discussion dealt with western conditions; but although the west has had more reason to be interested in both legal and hydrologic questions pertaining to ground water than have the more humid sections of the country, such questions are of vital importance to many in New England and elsewhere along the Atlantic Coast. Even if in some cases the interests in the East may be superficially in surface waters only, ground water in motion is only water which was surface water and is on its way to an outlet where it will again be surface water; and "all water beneath the surface of the ground, after all, is purely and simply ground-water, moving according to certain well recognized laws of physics. There seems to be no scientific reason why an

[3] "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 753.

elaborate and expanding classification of ground-waters should be necessary."[79] Ground water and surface water being interdependent, and there being no scientific justification for dividing the former into the legal classifications which have only resulted in confusion, it follows that every one who is directly or indirectly interested in water (and who is not?) should be concerned with the problem of ground-water administration. David G. Thompson's suggestions[80] as to initiating action are most apt. He not only directs attention to the necessity of conducting discussions on a broad basis, without restrictions along lines of location or of use or misuse of ground-water resources, but also lists other associations which include members interested in this condition, referring finally to the legal profession, since "when once the majority of the technologists agree, the position of the lawyer is to show not what the desired result shall be, but how it can be brought about."

[79] Discussion by D. G. Thompson of "Administrative Control of Underground Water: Physical and Legal Aspects," by Harold Conkling, *Transactions*, Am. Soc. C. E., Vol. 102 (1937), p. 809.

[80] *Loc. cit.*, p. 818.